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OF THE

AMERICAN CONCRETE INSTITUTE

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The ACI JOURNAL technical pages are a medium for reporting what is of current interest in the concrete field; in research, design, construction, maintenance, or manufacturing. Members and readers are urged to "have their say" as a means of sharing knowledge and experience; time and changing interests demand that new names and new ideas take their places beside the old. Contributions describing new materials and techniques or new applications of familiar practices are welcomed by the Institute.

To guide the flow of material the Board of Direction has assigned to the Technical Activities Committee the selection of papers, committee reports, discussions, and other contributions for JOURNAL publication. To assist in their technical appraisal a corps of experts in various segments of the field volunteer their services. From the technical standpoint, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

General considerations by TAC are whether the contribution will be of interest and service, will fit into a well-balanced publications schedule, and can be published at reasonable expense to the Institute.

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In ACI Journal next month

In "Strength of a Concrete Slab Prestressed in Two Directions" by A. C. Scordelis, K. S. Pister, and T. Y. Lin, the elastic behavior and ultimate strength of a full size concrete slab prestressed in two directions were investigated. The slab was supported only at the four corners, simulating a lift flat slab. It was loaded uniformly by means of air pressure in plastic bags. Experimental deflections and strains were checked against the classic elastic theory. Observed ultimate strength was compared to that obtained by the crack-line theory as applied to prestressed slabs.

Excavation and concreting problems encountered in tunnel construction near Santa Barbara, Calif. are described in "Lining of the Tecolote Tunnel" by E. R. Crocker. The principal problem was inflows of water up to 9000 gal. per min at temperatures which reached 117 F. Methods of overcoming the problem are described, and information on concrete materials, mixes, and placing procedures is presented.

"Theory of Stresses Induced in Reinforced Concrete by Applied Two-Dimensional Stress" with an appendix on application to shear strengths of shallow concrete beams is by Bruce H. Falconer. The theory of the main paper is applied to a consideration of the strengths of normal or shallow reinforced concrete beams, in which the longitudinal reinforcement is customarily located only near the top and bottom of the cross sections. The development follows logically from the main paper, without further basic theory, but does involve the neglect of possible influences of a rapidly changing stress field.

"Some Effects of Carbon Dioxide on Mortars and Concrete," by I. Leber and F. A. Blakey, discusses tests for determining the effect of gaseous carbon dioxide on the strength and shrinkage of mortars and concretes. Results from previous work appeared contradictory, but it now seems that strength is increased and shrinkage decreased if specimens undergo a period of curing in carbon dioxide immediately after demolding. The influences of carbon dioxide under pressure and of storage in air free from carbon dioxide were studied briefly.

James M. Prentis writes on the "Analysis of Inelastic Bending Stress in Concrete Beams." He proposes a method of determining both concrete stress and steel force based on strain observations and measured deflections. Sample calculations are given for a prestressed beam tested to failure.

Considerations for Construction of Subgrades and Subbases for Rigid Pavement*

Reported by Subcommittee I, ACI Committee 325

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SYNOPSIS

The approach to the study of subgrades and subbases† is motivated by economy in solving pavement problems created by the nationwide variation in soil types. The major problems of concern in the design of rigid pavements are discussed.

Soil classification and its limitations are discussed; the bases for the most popular systems are described briefly. The major properties that should be ascertained before a material is used as an integral member of the pavement system are considered and methods of test are referenced to standard specifications. Subbases are discussed from the standpoint of use in rigid pavement systems as well as their requirements for satisfactory service.

HIGHWAY AND AIRPORT SOIL PROBLEMS

Many of the problems encountered in highway and airport engineering are directly related to the engineering properties of soils. The economics of a proposed paving project, either highway or airport, is generally greatly influenced by the properties of the soil materials in place as well as the quality and availability of materials of construction.

The regional variation in soil types is well described in both geological and engineering literature (*e.g.*, reference 19). These variations are due mainly to differences in the origin of the formation of the various soils. Jenkins, *et al.*[11], pro-

*Prepared as a part of the work of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports. The report was submitted to the main committee (17 members) with 14 voting affirmatively and none negatively.

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†Subbase, when used under a rigid pavement, refers to a relatively thin "but stated thickness of a" layer of selected or specified material placed between the subgrade and rigid pavement at a specified (or stated) depth for such purposes as improvement of drainage, reduction in frost damage, prevention of pavement pumping, or moisture control of high volume change subgrade soils.

vide a good regional description of United States soils. Many publications provide the information needed to appraise soil conditions over a large area (*e.g.*, reference 14). Local variations at a specific project are best determined from a personal preliminary survey or exploration program.

The major problems of concern to the highway or airport engineer are related to inherent strength deficiencies of the soil in an area and to detrimental interaction between the soil and water under various climatic environments. It must, however, be realized that the problems to be discussed do not occur separately, but interact with each other in varying degrees.

Drainage

Adequate drainage of an airport or highway pavement is a prerequisite to design. Accumulation of water must be prevented to insure desired performance of the pavement. Pavement pumping, frost action, and landslides also are inherently associated with inadequate drainage.

Subsurface drainage can be effective in pervious soils and to intercept flow from water-bearing strata. In impervious soils, subsurface drainage is difficult at best, and may result in detrimental volume change. Surface drainage is always valuable and necessary and is a major step in retarding accumulation of water beneath the pavement.

Shrinkage and swelling

These phenomena represent only one portion of the general category of volume change of soils, and are associated with conditions of varying soil moisture contents. Certain types of highly plastic clays shrink and swell as their moisture contents vary. The natural moisture content at the time of paving appears to be related to the degree of such volume change, and, therefore, is a factor to be considered.[9] Differential shrinkage and swelling of subgrade materials will affect both the riding qualities and serviceability of the pavement.

Frost action

Frost action involves both freezing and thawing of moisture in a soil material, as well as the consequent changes of the soil conditions and its engineering capacities. Freezing of the soil moisture is associated with an attraction of water from a warm phase to a cold phase in a soil mass. Frost heave (volume change associated with ice segregation) requires an abundant supply of water, and is most detrimental when temperatures are lowered slowly and freezing occurs over a long period.

Spring break-up is the phenomenon reflected by low strengths in a subgrade soil or portions of layered systems caused by an accumulation of water as thawing occurs from the top down; the water is trapped by an underlying mass of frozen soil. Frost penetration of appreciable depths appears to be necessary for spring break-up to occur; frost heave is not necessarily an accompanying phenomenon, but oftentimes is also present.

Because of the presence of water the soil characteristics necessary include sufficient permeability as well as a relatively high degree of capillarity. The material commonly called "silt" best fits the foregoing description. Thus, silts are the most frost-susceptible soils known. The varved clays also are frost susceptible; they appear to contain the detrimental aspects of the silts as well as those of the clays, and are, therefore, doubly treacherous.

Discussion of these phenomena may be found in the literature.[8, 10, 12]

Landslides

Problems with landslides are common in almost all types of construction. Corrective actions for landslides have been described adequately in the literature;[3] mention need not here be made of them.

Stabilization

Use of locally available, low-cost materials for subgrades and subbases to improve subgrade support has important economic advantages. Poor materials may often be stabilized by mechanical means or chemical additives. Mechanical stabilization involves proportioning soil-aggregate mixtures in proper quantities to attain desirable dense gradations in the final product.

Admixtures such as lime, cement, calcium chloride, sodium chloride, and bituminous materials are used in chemical stabilization. Lime reduces plasticity and in some cases helps to bind the soil fraction of soil-aggregate mixtures. Portland cement has been widely used not only to change the plasticity but also as a binder for a wide variety of soils and soil-aggregate mixtures. Soluble salts such as calcium and sodium chloride have been used rather widely as admixtures for low cost roads; some experimental use has been made of calcium chloride for minimizing ice segregation in certain subgrades in "frost areas." Various bituminous materials may be used as waterproofing additives as well as binding media.

Compaction

Inadequate compaction of soils and rocks and combinations of them in embankments and subgrades generally leads to detrimental and unequal settlements, lack of subgrade support for the pavement, and even landslides. Modern earthwork practices require compaction under controlled and more or less ideal moisture contents to obtain the required density with the materials being placed.

Regardless of the method used, it is good practice to compact soils and rock-soil mixtures at moisture contents near those established by compaction and other laboratory tests. It has been found that a rather wide range on either side of the "optimum" moisture content can be used for silty-clay soils. However, silty soils are extremely sensitive to changes of moisture, and compaction at or slightly below the optimum moisture content gives best results. On airfields, where high tire pressure and heavy loads are anticipated, a field test section may be required to determine compaction requirements.

Compaction of broken rock mixed with soils and shales is difficult. Best results

are obtained when the rock is broken into small sizes, and when sufficient soil is available to fill completely the voids between the individual rock fragments. The behavior of a shale being compacted in an embankment will vary with the composition of the shale and its binding medium. Since most shales will disintegrate under freezing and thawing or wetting and drying, it is important that shales be compacted in at least a semi-disintegrated condition.

Adequate compaction of granular materials used as subbases is extremely important. Vibratory action of traffic will result in differential settlement of concrete slabs if the subbase is inadequately compacted. The coarse textured materials are most efficiently compacted by vibratory procedures. In compacting sands, however, use of water helps provide a well compacted material without a detrimental dust condition. A number of efficient and economical vibratory compactors are available.

Pavement pumping

Pumping occurs at or near joints and cracks and along edges of concrete pavement, and may be recognized from soil stains on the pavement and soil-water slurries at pavement edges. Three factors are essential to pumping: (a) free water must be present beneath the slab; (b) axle loadings must be heavy and frequent; and (c) the subgrade soil must be capable of going into suspension. If any of the foregoing factors is absent, pumping will not occur. Breaking of pavements will result from continued and uncontrolled pumping.

The free water immediately under the slab is largely surface water entering through cracks, joints, and at the pavement edges; water-bearing strata, high water table, or melting ice in the soil may also contribute free water. Pumping is most pronounced during and following extended periods of rainfall.

When a susceptible soil type and free water are present, the most important factor contributing to pumping is the passage of a large number of heavy axle loads. Most severe pumping occurs on pavements carrying the highest volume of heavy axles. Pumping is more pronounced when associated with the slower speeds of the heavy axle loadings.

The use of granular materials as subbases, when designed as a filter course[6] has been considered the best method of preventing pumping on new construction. Pumping may also be minimized by adequately designed load transfer devices which prevent faulting, and reinforcing steel at intermediate cracks.

Foundation support

Inherently weak materials such as peat bogs, muck deposits, and certain types of alluvial and other erratic deposits provide poor foundation support for rigid pavement. Since these materials often are incapable of supporting even a few feet of embankment, they must be studied in detail. Sampling for consolidation, permeability, and shear testing aid in the design of embankment slopes to eliminate possible foundation failure during or after construction of the embankment. Design techniques include flattening the embankment side slopes, slow construc-

tion to facilitate consolidation of the foundation soil without shear failure, and the use of "sand drains" and "permeable layers" to assist drainage during consolidation.

SUBBASES FOR RIGID PAVEMENTS

The supporting capacity of the subgrade material determines, to a large extent, the thickness of rigid pavement required to support loads moving across it. In general, the weaker and the more erratic the subgrade support, the greater will be the required thickness of the pavement. A nonuniform support requires that the slab be adequate to "bridge" soft spots; in effect a pseudo-beam is designed. Subbases and subgrade treatments tend to improve the uniformity of subgrade support for rigid pavements as well as to keep the subgrade intact and otherwise aid in overcoming the problems previously mentioned.

Subbase function

The subbase for a rigid pavement is a compacted layer of planned thickness placed immediately under the pavement and on the subgrade to serve one or all of the following functions:

(a) To provide uniform, stable, and permanent support for the rigid pavement.
(b) To prevent or reduce the damaging effects of frost action.
(c) To improve drainage and, when properly drained, minimize the accumulation of water under the pavement.
(d) To prevent pumping action at joints, cracks, and edge of pavement.
(e) To reduce volume changes in the subgrade and to minimize the detrimental effects of such volume changes on the concrete slab.
(f) To expedite construction.

Composition and physical properties

To perform the above functions, the subbase material should consist of hard, durable particles or fragments of granular aggregates, correctly mixed or blended with sand, clay, silt, stone dust, shell and/or other similar binding or filler materials, obtained from approved sources to provide a uniform mixture. The materials should be free from vegetable matter, or an excessive number of lumps of clay and other objectionable or foreign particles.

The subbase material should be well graded and, of the material passing the No. 10 sieve, not more than 25 percent should pass the No. 200 sieve. The percentage passing the No. 200 sieve should, also, not be greater than one-half that passing the No. 40 sieve. These gradation recommendations will, in general, provide a suitable subbase material, although exact specifications depend on the conditions under which the subbase will perform. The greater the weight and frequency of axle loads, the more stable and more resistant to pumping the subbase should be.

Stability

It is vitally important that subbase material be stable; the California bearing ratio test is often used to evaluate this property. Specific stability requirements may be found in the literature of the various agencies concerned with subbase design.[7]

Plasticity requirements

The portion of the subbase material passing the No. 40 sieve should have a maximum liquid limit of 25 and a maximum plasticity index of 6.

Nonfrost susceptibility

Where freezing of soil moisture under the pavement is probable, the subbase should be of a nonfrost susceptible material. Gradation requirements of nonfrost susceptible materials vary from a maximum of 8 percent finer than 0.02 mm for uniform sands to 3 percent finer than 0.02 mm for other nonuniform granular material.

Requirements to prevent pumping

Subbases composed of either undrained dense-graded materials or open-graded materials provided with adequate drainage have been found to be successful in preventing pumping.

Dense-graded subbase is usually constructed in an undrained trench somewhat wider than the pavement. This type, to be successful, must be so graded that it is practically impervious and must be compacted to maximum density. Surface water will penetrate into the subbase if compaction is inadequate or if the gradation is poor. If the gradation is poor, the subbase will have a high water holding capacity even though well compacted. Water in an undrained granular filter resulting from either improper gradation or compaction may lead to the expulsion of clear water at the pavement edge, causing a condition similar in appearance to pumping although no subgrade material is removed from underneath the pavement. Lack of proper compaction will lead to settlement of one or both slabs at joints as a result of traffic causing further compaction of the subbase.

Open-graded subbase must be properly compacted, and the gradation of this coarse material must be such that it prevents intrusion of subgrade soil into the subbase. The subbase permits adequate drainage without itself becoming clogged. It is designed so that water only, not subgrade soil particles, flows through its void spaces at a rate sufficiently low to prevent scour. The open-graded subbase must be provided with some system of drainage which can be maintained and kept open throughout the life of the pavement. Such drainage is provided by using shoulder drains, by constructing the subbase course the full width of the grade, or by longitudinal pipe underdrains near the edge of the pavement with porous material extending from the pipe to the subbase. In northern areas all shoulder drains tend to fill with ice during fall and winter and to remain frozen long after the subbase under the pavement has thawed each spring. As a result,

such drains may be ineffective during much of the season when conditions are most conducive to severe pumping.

Subbases 3 to 6 in. thick[6] have been used effectively to prevent pumping. Thicknesses greater than 6 in. are used to replace frost susceptible subgrade soils, minimize the effect of volume change, and to provide increased subbase-pavement strength, in addition to preventing pumping.

Load supporting value

For airport work, and where economical for highway pavements, the finished subbase should have a load supporting value such that when it is loaded in increments with a 30-in. diameter steel plate, it has a minimum modulus of foundation reaction, k, of 200 psi per in. of deflection. The subgrade modulus, k, should be evaluated by loading tests made directly on the finished subbase and/or trial sections of the subbase constructed in both cut and fill areas and at the transition between cut and fill areas.

EFFECT OF SUBBASE ON PAVEMENT THICKNESS DESIGN

Design of rigid pavements for the average highway is based largely on experience, past practices, and performance. The presence or absence of subbase material appears to be rather arbitrary, and there is little consistency in the use of a subbase in available design methods. The design of rigid pavements for relatively large loads requires an evaluation of the subgrade material as well as some consideration of frost action.

Corps of Engineers method

This method uses charts derived partially from the theoretical analysis by Westergaard[21] and partially from experience. The subgrade support is evaluated by plate loading tests and the value of k (modulus of subgrade reaction) is corrected for saturation, the worst condition possible. Using k, the flexural strength of concrete, and wheel loadings as variables, a thickness of rigid pavement may be obtained. Under normal conditions, or where freezing is moderate, or where the water table is at a large depth, a 4-in. minimum thickness of subbase designed as a filter is recommended.

When frost action is anticipated, modifications in the design for normal conditions are made. With an F-4 subgrade soil, the worst from the standpoint of frost susceptibility, a total thickness of subbase and pavement equal to the depth of frost penetration is used. Subgrade soils of less severe frost susceptibility are associated with a design modification that allows for a reduction in strength due to the frost action. A reduction in the modulus of subgrade reaction is made based upon appropriate charts. A thickness of subbase is used equal to the thickness of the slab designed assuming the reduced k value. Details of the procedure briefly described above may be found in the literature.[8]

Navy method

This method is similar, in general, to that of the Corps of Engineers, with modifications.[1] The modulus of subgrade reaction, k, is used and is also corrected for saturation. Design charts are again based on Westergaard's original analysis. A minimum CBR value of 30 after soaking and a minimum k of 200 lb per cu in. are required of subbase materials. The subgrade modulus, flexural strength of concrete, and loading are the variables used in the design charts available. The thickness of subbase depends on the results of loading tests made on trial sections during evaluation of the subgrade.

Portland Cement Association method

Again the Westergaard theory (as modified by Pickett) is used as the basis for rigid pavement thicknesses. The modulus of subgrade reaction is again determined as a factor for use in design charts; the value of k is not, however, corrected for saturation. A factor of safety must be used with the design chart and is applied to the strength of the concrete. A subbase is added to the pavement when non-uniform soil conditions exist or when detrimental volume changes or pumping is anticipated. The thickness of subbase is arbitrary and is recommended as one-half to one-third of the depth of frost penetration, 4 to 12 in. in cases of high volume change soils, and 4 to 6 in. in the case of anticipated pumping action. Details of the method are available in the literature.[4,5]

Civil Aeronautics Administration method

This procedure differs from those previously mentioned in that emphasis is placed on the results of studies of pavement performance under service conditions in the establishment of pavement and subbase thickness requirements. Rigid pavement thickness is obtained from a curve showing the thickness required for a specified single wheel loading. As in the other methods, this curve is derived from the application of the Westergaard analysis (modified in accordance with performance data) with the assumption of a standard set of conditions with respect to the quality and strength of concrete. In addition, a constant subgrade modulus is assumed as a result of the provision of variable subbase thicknesses which depend on wheel loading, subgrade soil, drainage conditions, and frost penetration.[2]

APPENDIX 1—EXPLORATIONS PRIOR TO DESIGN

Soil explorations may be divided into two general categories: (1) preliminary, and (2) exploratory. The first usually precedes the second; results of the preliminary survey are used to determine the extent and detail of the more costly exploratory surveys. Preliminary surveys may take the form of literature study of geologic and pedologic data, as well as interpretation of aerial photographs. Exploratory surveys include borings and geophysical methods.

Soil explorations are made to develop detailed information on the engineering characteristics of the soils and other materials to be encountered during construction. For highways and airport runways, explorations are generally confined to the

top 5 to 100 ft—that is, the surface materials. The data obtained through explorations and soil surveys are used by the design engineer in (a) establishing the final grade and alignment of the road or runway; (b) selection of fill materials for optimum use in embankments and subgrades; (c) obtaining subgrade information with respect to drainage requirements and the solution of potential problems of frost action, pumping of rigid pavements, and rutting of flexible pavements; (d) location of borrow as well as granular materials for possible use as subbases and bases; and (e) designing the component parts and the total thickness of the pavement structure. The information obtained through explorations and soil surveys is used also by contractors in estimating the cost of excavation and in planning earthwork construction. (Reference 23 has excellent references on this subject.)

APPENDIX 2—TESTS OF SUBGRADE, EMBANKMENT, AND SUBBASE MATERIALS

The use of the various classification systems necessitates performing mechanical analyses and Atterberg limits tests on soils to be evaluated. In addition, special tests are utilized in the study of special problems of design and construction.

In general, the following properties are often determined:

(1) Grain size distribution and other physical characteristics
(2) Plasticity characteristics
(3) Volume change
(4) Moisture-density relationship
(5) Strength
(6) Permeability
(7) Compressibility

In the evaluation of these properties, a series of standard and nonstandard soil tests has been devised. These tests are outlined below and their details may be found in the reference appended to each nonstandard test or in the appropriate specification of the standard test.

	Designation	
Classification tests	**AASHO**	**ASTM**
Mechanical analysis of soils	T 88-49	D 422-51
Liquid limit of soils	T 89-49	D 423-39
Plastic limit of soils	T 90-49	D 424-39
Specific gravity of soils	T 100-38	D 854-52
Shrinkage factors of soils	T 92-42	D 427-39
Field moisture equivalent	T 93-49	D 426-39
Sieve analysis of fine and coarse aggregate	T 27-46	C 136-46
Amount of material finer than No. 200 sieve	T 11-49	D 1140-50T

Design tests

Strength index tests
(a) Plate bearing tests—Reference 16.
(b) California bearing ratio test—Reference 16.

Consolidation tests—Reference 16.

Compressive strength tests—Reference 16.

Permeability tests—Reference 16.

Construction control tests

Compaction and density of soils—AASHO T 99-49, ASTM D 698-42T.

Many modifications of the standard test exist; details of many of the variations

may be found in the work of Lambe. [15]

Density of soils in place—AASHO T 147-49 and reference 16.

APPENDIX 3—SOIL CLASSIFICATION

With respect to highway and airport pavements the various systems of soil classification are similar in that basic information is obtained by means of surveys to determine the soil profile on the route or site and from laboratory tests on samples of the different layers of the soil profile. In general, the laboratory tests consist of mechanical analysis, liquid limit, and plasticity index determinations and the soils are grouped in accordance with the results of these tests. However, grouping is actually based on the known behavior of the soils and the test results serve as a means of identification. These systems of classification are generally completely satisfactory but additional testing may be necessary. (See list of soil test methods in Appendix 2 where special conditions are encountered.)

Classification systems

Classification systems of major importance to the highway and airport engineer are listed below and reference is made to other publications for complete details.

Bureau of Chemistry and Soils[24]—The system is based entirely upon soil texture, *i.e.*, grain size and grain size distribution.

Highway Research Board system[17]—This system is based upon grain size distribution, plasticity characteristics, and to some extent certain other properties upon which performance depends. It attempts to evaluate elasticity and volume change on the basis of an empirical relationship between various Atterberg limits. An empirical quantity known as the "group index" is used to evaluate clayey-granular and fine textured materials on the basis of plasticity characteristics.

Civil Aeronautics Administration system[2]—This system separates coarse-textured soils from the fine-textured soils on the basis of mechanical analysis. Coarse-textured soils are subdivided into groups according to grain size distribution, liquid limit, and plasticity index. Fine-textured soils, containing more than 45 percent combined silt and clay, are grouped in order of increasing liquid limits and plasticity indexes. For pavement design the soil groups have corresponding subgrade classes based on the performance of the particular soil as a subgrade for pavements under different conditions of drainage and frost.

Unified soil classification system[20]—This system is coming into wide use because it is comprehensive and well adapted to pavement design. Coarse-textured soils are delimited and differentiated according to grain size and grain size distribution. The fine-textured soils are separated by means of plasticity characteristics in conjunction with field descriptions; this latter item allows for easy recognition of the presence of organic materials. The system has good value as an engineering classification method.

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For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

Some Implications of Recent Diagonal Tension Tests*

By PHIL M. FERGUSON†

SYNOPSIS

On the basis of the observed cracking which occurs when a beam fails in diagonal tension without stirrups and away from complicating loads and reactions, a hypothesis of unrestrained‡ failure is developed. This hypothesis assumes each step in the cracking pattern as a tension failure which can be rationalized in terms of the conventional combined stress formula.

Failures of beams somewhat restrained by reactions or loads, of beams of large shear span, and of footings are each discussed in terms of this hypothesis.

Finally, two exploratory series of tests are reported. The first investigates the effect of extra or multiple loads and suggests that higher shear strengths are available near supports. The second series indicates that much of the increased capacity associated with small shear spans is lost if the loads are applied as shears over the depth of the beam or if the reaction is applied as a shear.

INTRODUCTION

Experimental study of diagonal tension in reinforced concrete beams has resulted in important new concepts, but these are accompanied by a new awareness of gaps in our knowledge. Beyond the proven facts there exist indications which are less conclusive, evidence which might be designated as the implications of recent research.

This paper summarizes some of these implications in the form of an hypothesis which traces out the observed failure pattern step by step. Each step is rationalized or explained in terms of the standard theory of combined stresses. It is implied that before long one may, at least in research, find it possible to use the theory of combined stresses more constructively in connection with diagonal tension and thus to depend less upon purely empirical relationships.

After discussing various cases of diagonal tension failure in terms of this hypothesis, the paper also reports two short series of exploratory tests at the Univer-

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‡The term "unrestrained" is used to designate a failure generally removed from the influence of local stresses around loads and reactions.

sity of Texas which seem to indicate the need for more emphasis on combined stress relationships.

The entire presentation relates specifically to simple span beams without stirrups, but the same analysis would seem to be applicable to continuous spans as well.

Recent investigations

Current American interest in reinvestigating diagonal tension and in reappraising conventional methods stems in considerable measure from Richart's 1948 footing test paper.[1] He found that conventional calculation methods could not be reconciled with his footing tests. Yet he was unable to suggest a more consistent method of analysis.

In 1951 Clark reported,[2] among other items, that shear strength was sensitive to changes in the shear span or a/d ratio (Fig. 1). The shear span is the distance from reaction to load for a one-point or two-point load arrangement. For a small shear span, with $a = 1.2d$, the shear (diagonal tension) capacity was over two times as large as for $a = 2.3d$.

In 1953 J. Neils Thompson and the writer pointed out[3] that diagonal tension strength did not increase directly with f_c'; in fact, it increased rather slowly for an f_c' over 3000 psi. (The 1956 ACI Building Code limits allowable shear to be carried by the concrete for higher f_c' values to the 90 psi permitted for $f_c' = 3000$ psi.)

This paper also presented the opinion that the a/d or shear span effect is a local loading effect, specifically, that the increased resistance to diagonal tension with small a/d ratios is produced by the vertical compressive stresses set up over reactions and under concentrated loads.

In 1953 Hognestad showed[4] that the results of Richart's footing tests could be correlated: (1) by introducing a factor ϕ, for the ratio of the column load causing failure in diagonal tension to the load that would cause a bending moment failure; and (2) by calculating the shear v_2 at face of column rather than the usual v_1 at a distance d from the column. This v_2 value he found to be:

$$v_2/f_c' = 0.035 + \frac{130}{f_c'} + \frac{0.07}{\phi}$$

It should be noted that v_2 here is much in excess of the shear strength ordinarily associated with beams.

In recent years the University of Illinois has taken the lead in shear studies. Zwoyer and Siess in 1953, Moody, Viest, Elstner, and Hognestad in 1954 and 1955, and Laupa, Siess, and Newmark in 1955, have all reported[5,6,7] significant contributions which included both new test data and a reanalysis of all other known data. They describe diagonal tension failure as a matter of: (1) diagonal tension cracking, followed by (2) a readjustment of internal stresses, (3) usually the capacity for greater load, and (4) a final failure in bending under special conditions that have been termed a shear-compression failure (Fig. 2). This analysis constitutes an impressive step forward in our understanding of diagonal tension, especially for small values of a/d.

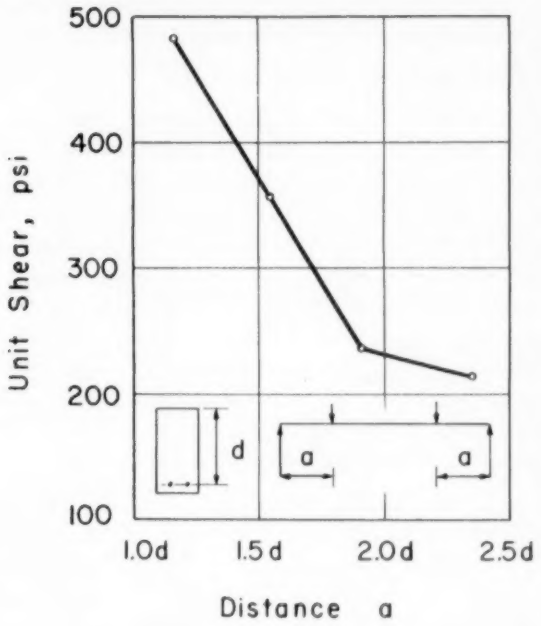


Fig. 1—Variation in shear strength with shear span (Clark's tests)

HYPOTHESIS OF UNRESTRAINED FAILURE

Since 1947 the writer has watched over 130 diagonal tension failures in the laboratory, a majority of these with larger rather than smaller a/d values. The above shear-compression theory alone has not adequately described the complete failure process observed in many of these tests. The diagonal crack of Fig. 3 represents such a failure, nearly complete and very little distorted by the final failure in bond. This paper extends the shear-compression failure idea into a more general hypothesis of the failure mechanism needed to explain this type of cracking.

The hypothesis states that it is possible to describe an unrestrained diagonal tension failure as a series of tension crack developments, each of which can be explained in terms of the standard theory of combined stresses. (The term "unrestrained" is used to designate a failure generally removed from the influence of local stresses around loads and reactions.) The hypothesis envisions the following steps in the development of this unrestrained failure, as indicated schematically in Fig. 4: (1) an initial diagonal crack forming near middepth and stopping within the compression area at 1a, and in the tension area somewhere in the zone marked 1b; (2) a somewhat flatter extension of this crack in the compression zone at increasing loads; (3) a general cracking in the zone around the steel, which may develop simultaneously with (2); (4) a sudden final failure by an extension of the tension crack marked 4a or the shear-compression failure marked 4b, with a secondary failure in splitting and bond at 4c. In some cases the failure in 4c can develop into the primary failure, as in the beam of Fig. 3.

Each of these phases will now be examined in some detail in an attempt to explain it in terms of conventional combined stress theory.

Initial diagonal cracking

The first diagonal crack may start in either of two ways.

It may form where moment cracks do not exist. The tensile bending stresses near the neutral axis are then small and the cracks develop just below middepth [Fig. 5(a)]. The tensile cracking stress t_o must then be approximately equal to the maximum unit shear v_o given by VQ/Ib or, closely enough for rectangular beams, by $1.5\ V/bh$.

In other cases, especially when a/d is large, a diagonal crack may develop from a moment crack, evidencing the shear effect by turning from its vertical direction as shown in Fig. 5(b). Such a crack rarely becomes critical before increased loads cause it to extend back toward the steel, as indicated by the dotted line in the figure. Since shear cannot cross the crack, the total shear, except that on the bottom bars, must be carried in the small depth y, a condition not recognized in either the relation $v = VQ/Ib$ or $v = V/bjd$. The real unit shear above the crack must be greatly increased. Hence it appears reasonable that such a crack should start at a smaller total shear than would be required in the uncracked section.

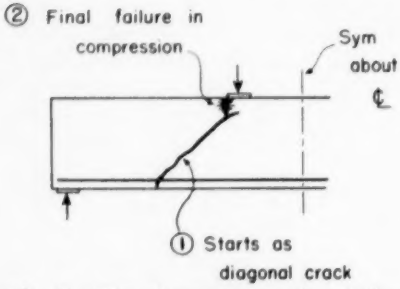


Fig. 2—The shear-compression failure

Either type of crack, once started, will penetrate above the neutral axis into the compression zone.* Consider the stress condition at the top of a crack which, say, has opened over only a $0.2h$ height (Fig. 6). Since only 80 percent as much shear area remains, the shear here must be at least $v_o/0.8 = 1.25\ v_o$ where $v_o = t_o$ = the original cracking stress. This increased shear must lead to continued

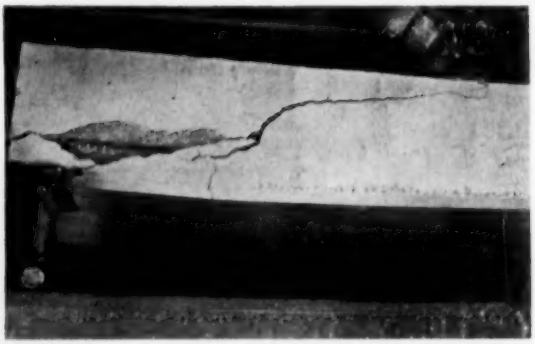


Fig. 3—Diagonal tension crack just prior to failure. Actual failure in bond at left. (Picture is inverted to place crack in usual orientation)

*The University of Illinois research group have deduced this empirically but appear to consider that the neutral axis actually rises to stay above the crack.

cracking until a sizable compression is encountered.

If the compression stress required to stop the diagonal crack is designated $-f$ (negative for compression), its approximate value can be established from the relation:

$$t = \frac{-f}{2} + \sqrt{\left(\frac{-f}{2}\right)^2 + \left(1.25v_o\right)^2} = t_o = v_o$$

This yields the value, $f = 0.57\ v_o$. Thus the crack will not stop until it reaches an $f = 0.57\ v_o$ in compression; and if this creates a crack longer than $0.2h$ in height, v will be still larger and f must likewise be greater to stop the crack. This cracking in the compression zone will be at a slope flatter than 45 deg. For example, $f = 0.57\ v_o$ indicates a 37-deg slope and $f = 1.5\ v_o$ a 27-deg slope, since $\tan 2\theta = 2v/f$.

The analysis suggests that at a point of inflection the formation of a diagonal crack should lead to immediate failure because there would exist no compression capable of stopping its progress. However, in tests run by the author where this might have occurred, the diagonal cracks have all started in a moment region, as in Fig. 5(b), instead of at the point of inflection.

Extension of diagonal crack

Why diagonal cracks, once started, do not *always* continue to the tension steel, is not clear. They frequently do so, but on other occasions some further load increment seems to be necessary to bring this about. Once the crack has thus extended to the steel, a considerable redistribution of bending stress has also occurred, as pointed out by the University of Illinois group and as shown in Fig. 7. The free body of Fig. 7(b) assumes the steel can resist a negligible portion of the external shear.* Summation of moments about A indicates that both T at B and C at A are determined by the moment at A. Then $C = Vx/jd$, where jd is probably greater than that usually calculated. If the uncracked depth is y, the average $f_c = C/by = Vx/(byjd)$.

Due to the crowding of the compression from the cracked zone into this restricted area, f_c is probably more nearly trapezoidal than triangular. For

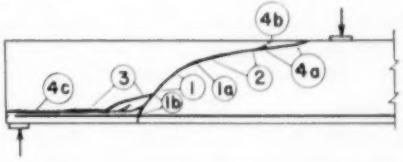


Fig. 4—Unrestrained diagonal tension failure. Sections of crack numbered according to hypothesis of failure

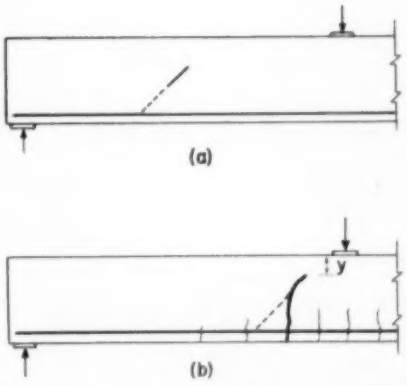


Fig. 5(a)—Initial crack in zone of small moment (b)—Initial diagonal crack where moment is large

*This assumption needs further study.

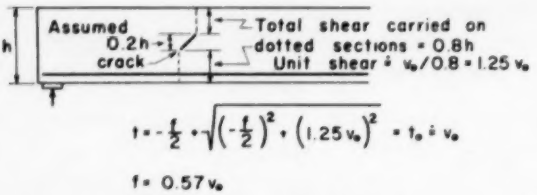


Fig. 6—Effect of a small crack on shear stress

small values of y, the stress probably becomes nearly uniform.

The shear stress is still less definite, but the average $v = V/by$. If one considers the stresses just above the crack as being the average shear and half of the average f,

$$t = -\frac{Vx}{4byjd} + \sqrt{\left(\frac{Vx}{4byjd}\right)^2 + \left(\frac{V}{by}\right)^2} = \frac{V}{by}\left[-\frac{x}{4jd} + \sqrt{\left(\frac{x}{4jd}\right)^2 + 1}\right]$$

If, for example, $x/d = 3$ and $j = 0.9$,

$$t = \frac{V}{by}\left[-\frac{3}{4 \times 0.9} + \sqrt{\left(\frac{3}{4 \times 0.9}\right)^2 + 1}\right] = 0.47V/by$$

For $y = 0.3d$, which would mean a crack only slightly above the neutral axis of a rectangular beam,

$$t = \frac{0.47V}{b(0.3d)} = 1.57V/bd$$

Since this tension t is in excess of the original cracking stress $t_o = v_o = 1.5V/bh = 1.36\ V/bd$ (for an h of $1.1d$), it indicates that cracking would certainly continue. On the other hand, if one assumed f as being the average f instead of half the average, t would become only $0.28\ V/by = 0.93\ V/bd$. Also, an x value greater than $3d$ leads to a larger f, less diagonal tension, and less probability of an extending crack. The observed fact is that generally the crack does stop and that more load is required to make it continue. If the extra load is provided, the crack continues by increments but at a flat slope. For $f = Vx/byjd$, $v = V/by$, $\tan 2\theta = v/(f/2) = 2jd/x = 2(0.9)/3 = 0.6$, leading to $2\theta = 31$ deg or $\theta = 15.5$ deg. The observed crack flattens even more in many specimens, indicating that the critical stresses are not exactly the average values assumed, or that other forces are present. For example, the vertical shear, concentrated in the shallow depth y, must create an additional vertical tension stress at the end of the crack. Despite much uncertainty as to details, it is

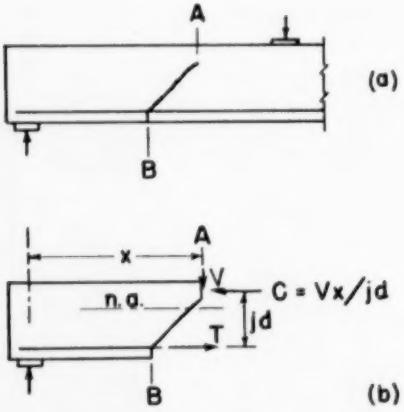


Fig. 7—Redistribution of bar tension caused by a diagonal tension crack

suggested that calculations such as these may assist in developing an understanding of the very flat cracks observed in the compression zone.

As the crack progresses under increasing V, y decreases, and the diagonal tension t is thereby increased until ultimately a progressive failure occurs on a plane about like that shown in Fig. 8. Under some conditions the increased shear and compression might lead to the shear-compression failure shown by the dotted lines.

Bond and shear near steel level

At the same time the crack is moving upward, some weakness usually develops at the level of the steel. Fig. 7 indicates that the tension in the steel at B is calculated with the moment at A and thus is as large as that usually calculated at a vertical section through A. Accordingly, as the crack moves upward and x increases, T at B is also increased, sometimes to as much as 150 percent of the design value, possibly even more. The increased tension produces exceedingly high bond stresses just to the left of B, the usual high bond stress at any crack being increased by the extra tension that must be picked up here. Since $vb = u \Sigma o$, the shear stress also becomes large and cracking as in Fig. 9 develops, often followed by some splitting of the concrete from bond and a progressive development of the failure zone toward the support. The final failure of the beam in Fig. 3 was of this kind.

In many cases there is also an indication that the shear carried across the crack by dowel action of the bars increases as the diagonal tension crack at the top of the beam flattens out and the depth y becomes small. Dowel action introduces a vertical tension which adds to the splitting and cracking forces.

HYPOTHESIS APPLIED TO SPECIAL CASES

In the light of the above hypothesis of the mechanics of an unrestrained failure, several special cases will be considered.

Restrained failures

The effect of a simple beam reaction under the end of a test beam is quite important. Sizable vertical compression stresses $(-q)$ are set up in this zone (Fig. 10, shaded area) which reduce the potential diagonal tension due to shear in this neighborhood in accordance with the relation

$$t = -\frac{f+q}{2} + \sqrt{\left(\frac{-f+q}{2}\right)^2 + v^2}$$

The presence of the compressive force q reduces t a little where f is tension $(+)$ and a large amount where f is compression $(-)$. Accordingly, in a constant shear section, stress conditions are more favorable to cracking where q is zero. When the load is applied far away from the reaction, a diagonal crack thus avoids the the vertical compression zone and is not apt to form closer, say, than $1.5d$ to the reaction. If the load is applied close by, *i.e*, a small a/d, the load also provides some vertical compression forces adjacent to the shaded area of Fig. 10. A higher

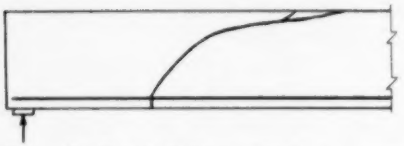


Fig. 8—Final failure crack in unrestrained zone

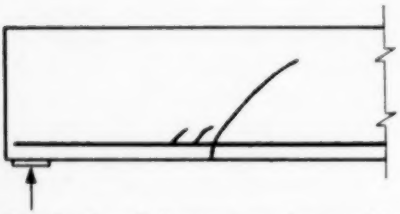


Fig. 9—Local diagonal tension cracking caused by increased bar tension and increased bond

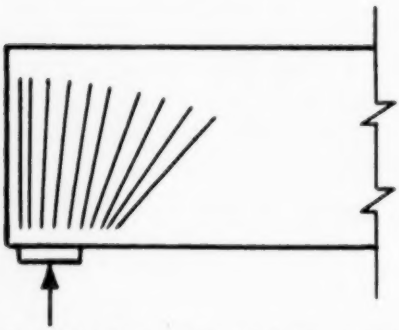


Fig. 10—Zone of vertical compression stresses over a simple beam reaction

unit shear is required to create the first diagonal crack in this vertical compression zone.

The vertical compression over the reaction is high enough also to strengthen a beam materially against combined bond splitting and diagonal tension cracking along the steel.

Next consider the local effect of a load applied on top of the beam at a point such that $a = 2.5d$. After the initial crack forms, it probably stops at A, as shown in Fig. 11, close to the load. The vertical compression under the load prevents or retards the formation of the flat diagonal tension crack pattern discussed above thus enabling the beam to carry a further increase in load. It will finally fail by crushing the depth y in compression in a moment type of failure, but with a moment capacity smaller than usually expected in approximately the ratio that y is less than kd. This is essentially the shear-compression failure envisioned by the Illinois group. A variation of this failure sometimes occurs with a nearly flat extension of the crack, as shown in Fig. 11 (b). It requires a large shear to make this crack develop and whether it differs in ultimate resistance from the other type failure has not been established. Either failure gives a shear capacity much in excess of that which would exist in a totally unrestrained shear failure. This is the reason that Clark and others have found higher shear strength for small a/d ratios.

Failures at large a/d values

For large a/d values in beams with only moderate reinforcing steel, a true moment failure may occur without the shear at any time reaching the v_o required to start a diagonal tension crack. When the a/d value is reduced, *i.e.*, shorter shear span, or the A_s is increased, moment failures can occur only after larger shears develop. Under either condition a diagonal crack may start prior to moment

failure, as in the case of the totally unrestrained shear failure.

If this diagonal crack occurs at a section where the moment is already near capacity in compression, it is possible that the intrusion of the diagonal crack above the neutral axis may so reduce the compression area as to bring about a nearly instantaneous compression failure, that is, a true shear-compression failure immediately upon cracking. More generally a beam is under-reinforced, compression is not so critical, and the initial diagonal tension crack stops. A subsequent increase in load develops the flat crack pattern to failure, as discussed in the development of the unrestrained failure hypothesis. This failure must occur at a smaller shear than that causing a true shear-compression failure, since the alternate possibility of a crushing failure above the crack always exists. Such cracks do not necessarily progress all the way to the load point and in this respect differ from the failure envisioned by the Illinois group.

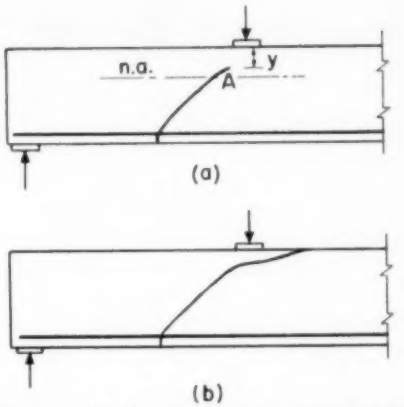


Fig. 11(a)—A load may stop a diagonal crack and lead to a shear-compression failure. (b)—A variation from the shear-compression failure

It would follow that for unrestrained shear failure (large values of a/d) the effect of the percent of A_s would indeed be small. Small A_s values might lead to moment failure before a diagonal tension crack could form; but, if a diagonal tension crack once forms, a further increase in A_s would have little effect on the load at which this crack would start or would develop to failure. One thus returns to the idea that, for large a/d ratios, the diagonal tension resistance is primarily a function of shear rather than moment.

More data are needed in this area. Values of a/d less than 4 or 5 are not large enough to give this full unrestrained failure condition.* The run of the flat part of the failure curve makes the diagonal crack itself use up something like a $2d$ length of beam or more. If a load on top of the beam interferes with this crack, it tends to increase the shear capacity of the beam. For such a load to be effective, however, it must be large enough to prevent the continuation of the diagonal crack and thus force a shear-compression type of failure. It appears that shear strengths of simple beams are lowest for beams with a large shear span or large a/d ratio.

Footing failures

The hypothesis is consistent with Hognestad's findings on footings. In footings, because the total width which resists shear increases as the square of the distance

*In an earlier paper, the writer expressed the opinion that $a/d = 3.4$ would be adequate; but the flat failure crack calls for more room.

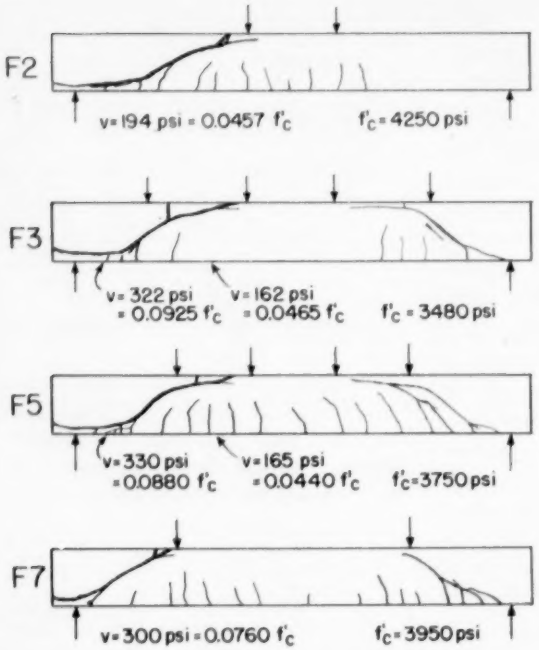


Fig. 12 — Comparison of beams with one and with two loads on each end

from the center of the column, the shear resistance increases rapidly as areas farther from the column are considered and leaves the weakest section at or quite near the face of the column. The steep (45-deg) part of the diagonal tension crack must first form there. If the section is already highly stressed in moment, the encroachment of the diagonal tension crack into the compression zone helps to bring failure quickly. If the moment is low at the time the crack forms, the footing continues to carry more load until the shear-compression failure develops at the column face. Thus it is logical that Hognestad found better correlation of shear values when a factor ϕ involving the moment capacity of the footing was included.

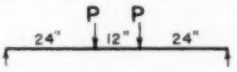
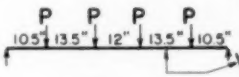
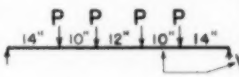
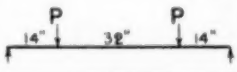
EXPLORATORY TESTS

Effect of multiple loads—tests

The results of four rectangular beam tests will now be noted. Each beam was approximately 4 in. wide, 8.25 in. deep over-all, 5 ft. 6 in. long, and was tested on a 60-in. simple span with two or with four symmetrical loads. The concrete was made with ¼-in. maximum sized aggregate. The reinforcing steel was two #5 rail steel bars satisfying ASTM A 305. End anchorage of bars was provided by welding the bars to a 2 x 3.75 x 0.5-in. steel plate at each end. All beams failed in diagonal tension under the loads and stresses indicated in Table 1.

Fig. 12 shows sketches of the four beams, their loading, their failure cracks, and the shears at failure. Attention is called to the failure shear of 0.0457 f_c' for

TABLE 1—LOADS AND STRESSES AT FAILURE IN DIAGONAL TENSION

Beam No	b, in.	d, in.		Max V, lbs	f'_c, psi	v, psi	v/f'_c
F_2	3.97	7.44		5000	4250	194	0.0457
F_3	4.06	7.22		8290	3480	322	0.0925
				V_2=4160	3480	162	0.0465
F_5	3.75	7.11		7690	3750	330	0.0880
				V_2=3860	3750	165	0.4400
F_7	4.00	7.25		7630	3950	300	0.0760

beam F2. The crack pattern indicates that this is nearly, but not quite, an unrestrained shear failure. When an additional equal load is placed within this 24-in. shear span, as in beams F3 and F5, it should be noted that the failure load is approximately doubled; but the failure shear adjacent to the central loads (noted as V_2) is nearly unchanged at 0.0465 f_c' and 0.0440 f_c'. Both these beams failed in an unrestrained failure pattern and both appeared near failure at the opposite end under the same crack pattern.

The vertical compressive stresses under the exterior load, especially in beam F3, delayed the opening of the first diagonal crack:

Beam	Shear at first diagonal crack
F2	Between 3500 and 4000 lb
F3	Between 5000 and 6000 lb
F5	Between 3500 and 4500 lb
F7	Between 4500 and 5500 lb

However, it should be noted that on the inner plane, which seemed to control final failure, these cracking loads on F3 and F5 correspond to lower shears than on beam F2.

The ultimate strength of beam F7 indicates that beam F5 must have been near failure on the outer shear plane as well as on the inner plane where it did fail.

Such tests lead one to suspect that with multiple loads much greater shear capacities can exist than practice now considers to be safe. These beams show that the shear can be doubled for a short length near the support without distress. One might expect this from Clark's tests and from the shear-compression theory of failure.

Significant savings are possible, especially on structures such as simple span highway girder bridges, when the limitations of this element of strength are fully known. However, even the proposed allowable u of 90 psi gives only the skimpy factor of safety of 1.80 on the inner panel of beam F3 and 1.83 on beam

F5. The second section following this also indicates that there are still unexplored factors which in some cases can cancel out much of this indicated advantage of higher shear capacity near the support.

The maximum moment in beams F3 and F5 was some 20 percent more than in beam F2 and was resisted by a concrete strength 12 to 18 percent lower. This would seem to make it difficult to correlate these failures on a moment basis.

Magnitude of the a/d effect

Beam F2, above, had a shear span of $3.23d$ and failed at $v = 0.0457f_c'$. Beam F4, discussed in the next section, had a shear span of $1.35d$, being both deeper and shorter than F2. It failed at $v = 0.196f_c'$, at over four times the shear coefficient found for beam F2. A small part of this difference probably lies in the fact that f_c' was 4250 psi for beam F2 and 3070 psi for beam F4, the lower f_c' always giving a larger coefficient. The major difference is the effect of the a/d ratio, an effect already well established. Fig. 13 indicates that the beam F2 failure was nearly, but not quite, unrestrained. The flat portion of the failure crack developed under the load but the final failure indicates restraint from the load. The figure also shows the steeper crack and deeper shear compression failure section accompanying the small a/d ratio of beam F4.

Effect of manner of supporting load—tests

Believing that much of the favorable effect of small shear span was due to the vertical compression developed over the reaction and under the load, the author tested beams S1, S2, and F4. Beam F4 was 4 in. wide, 10 in. deep to steel, 42 in. long, and reinforced with two #5 bars with welded end anchor plates. This beam was loaded symmetrically on a 39-in. simple span with a shear span of $1.35d$, and failed at $v = 0.196f_c'$, for $f_c' = 3070$ psi.

Beams S1 and S2 were of a different form, purely a laboratory shape as shown in Fig. 14. The cross members were a device for applying loads or reactions as shears on the sides of the beam rather than as loads at the top or bottom surface of the beam. This is the way loads are transferred when beams frame into a girder. If loads are thus applied to the sides of the beam as shears and not on *top* of the beam, the vertical compression stresses are much reduced, especially near the top of the beam. Without the vertical compression required to stop the diagonal tension crack, there is nothing to stabilize the beam for a shear-compression failure.

In testing beam S1, no use was made of these cross members and the ultimate v was $0.186f_c'$ for $f_c' = 3420$ psi, about the same as for beam F4. The failure crack is shown in Fig. 15, with near failure at the opposite end also. For beam S2 the loads were applied on the two center cross members as symmetrical loads 15 in. apart (7.5 in. on each side of the beam axis) but the reactions were kept directly under the beam. Fig. 16 shows that the diagonal crack was forced to a steep inclination but that it turned into a flatter crack between the load points. A shear-compression failure did not occur and the failure shear was only $0.0735f_c'$, or about 39 percent as large as with loads applied directly to the top of the beam.

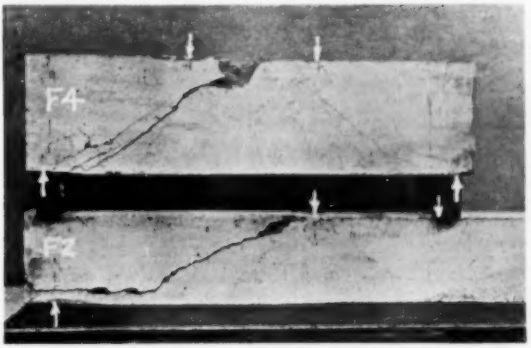


Fig. 13—Comparison of failure cracks on short shear span (F4) with those on longer shear span (F2)

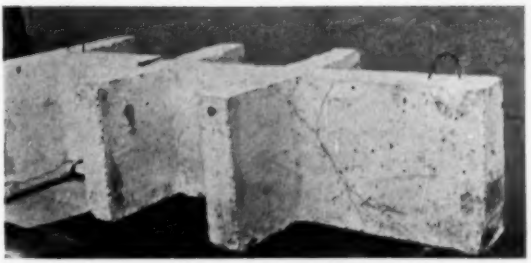


Fig. 14—Special beam with cross members to apply loads and reactions as shears on the sides of the beam

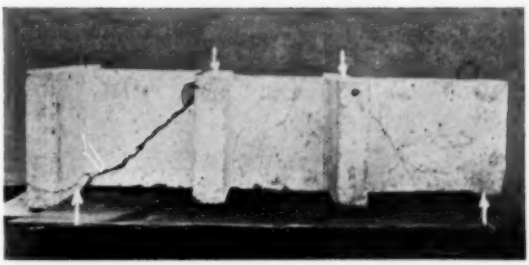


Fig. 15—Typical shear-compression failure on beam S1 with loads on top of the beam and reactions on bottom of beam

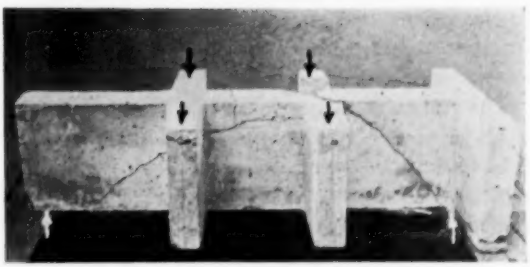


Fig. 16—When loads are applied as shears on the side of beam (S2) the shear strength is only 39 percent of beam S1 in Fig. 15

A similar beam, S4, was loaded directly on top, as in beams S1 and F4; but it had the reaction at one end applied through the cross member, thus producing a supporting shear along the sides or end of the beam. In this case the top loads

restrained (prevented) the formation of a diagonal crack under the load, but the reaction permitted a flatter crack to form and break out the end above the steel level (Fig. 17). The shear carried at failure was $0.0727f_c'$, or only 38 percent of the shear-compression value of beams S1 and F2. This is almost the identical value obtained from beam S2 with the loads applied as shears.

When both reaction and loads were brought in through the cross members as shears, the diagonal crack, once it had formed, had minimum restraint. Beam S3 loaded in this fashion failed as shown in Fig. 18. The author had expected a flatter failure crack in the vicinity of the loading cross members. However, the failure crack was almost the same as that of beam S4 which had only the reactions brought in by shear from the cross member. The ultimate v was $0.0596f_c'$, about 18 percent lower than for beams S2 and S4, and only 31 percent of the shear-compression failure strength.

These beams showed clearly that both the direct bearing of the loads on top of the beam and of the reaction under the beam are important elements in the higher shear strengths realized by the shear-compression failure in this short shear span. Loads transferred by shear, centering near middepth of the beam, fail to provide anything like the same protective vertical compression at the critical zones to stop the spreading diagonal cracks. Before the economy of larger shear design stresses near the supports can be realized, more information is needed relative to questions such as the effect of type of loading and support. Shear strength is not simply a matter of a/d ratio; it is more a matter of combined stresses.

Beam F6 represents an attempt to determine whether still lower resistance would be developed when the load was applied on cross members entirely within the lower half depth of the beam. Fig. 19 shows the flat cracks that were developed. This failure was entirely sudden. A diagonal tension crack, once started, had nothing to stop it. As a matter of fact, such a loading produces a vertical tension over the loading zone which accelerates the crack development. The shear developed was slightly smaller than on the other beams, $v = 0.0554f_c'$, or 29 percent of the shear-compression failure value. The cracks seem to be about the same at the end where the reaction was directly applied, starting close enough to the loads not to be much influenced by the reactions.

The results of beam F6 might well lead to doubt as to whether up-turned spandrel beams are not sometimes weakened in diagonal tension resistance when a large part of their load is brought in near the bottom of the beam.

The results of this series of beams are shown diagrammatically in Fig. 20. All of these results could be predicted qualitatively, and possibly quantitatively, by considering the conventional formula for combined stresses. The effect is exaggerated here because the small a/d, with loads on top of the beam and reactions on the bottom of the beam, develops such heavy vertical compressive stresses that the resistance to shear is unusually large—over four times the unrestrained shear value. Even the reduced strengths reflect some of the delayed initial cracking that results from the small a/d, value. These strengths were all higher than the $0.0457f_c'$ value found for beam F2, a beam showing nearly an unrestrained failure.

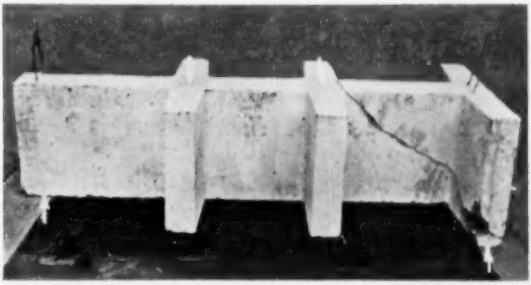


Fig. 17—Beam S4 with shear reactions at one end was only 38 percent as strong as beam S1 in Fig. 15

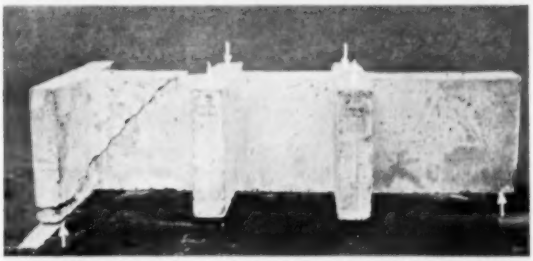


Fig. 18—With both loads and reaction applied as shears, beam S3 was only 31 percent as strong as beam S1 in Fig. 15

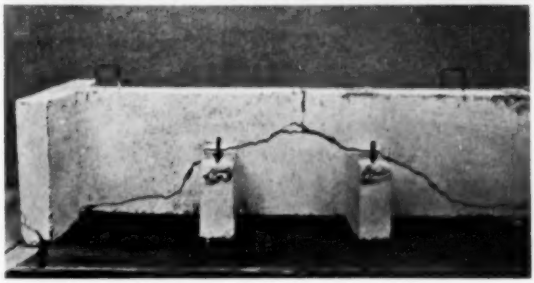


Fig. 19—Beam F6 with loads applied below middepth is subject to vertical tensile stresses that lower its strength

CONCLUSIONS

1. This hypothesis of failure emphasizing the mechanism of unrestrained diagonal tension failure seems to offer promise as a tool for visualizing the general aspects of diagonal tension.

2. The combined stress formula has some possibility of development into a useful calculation procedure, especially for research.

3. The hypothesis would seem to predict that: (a) In a simple span, the minimum shear strength will accompany a large a/d ratio. This will be an unrestrained type of failure. (b) The minimum shear in (a) might be lowered somewhat in the case of a beam about to fail in compression, where a diagonal crack might trigger a premature moment failure.

4. Under some conditions, such as loads applied directly on top of the beam and reactions directly on the bottom, design advantage could be taken of the higher shear capacity near supports.

5. When loads are brought in as shears or reactions are taken out as shears,

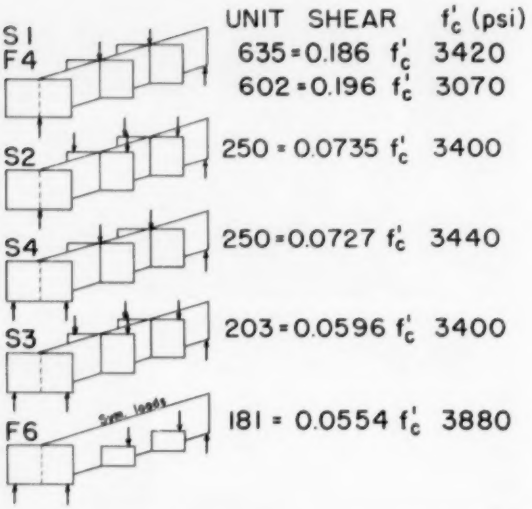


Fig. 20—Effect of loading conditions on shear capacity, constant a/d = 1.35

a large part of the extra strength associated with small a/d values is lost.

6. The concepts of combined stresses from engineering mechanics should be used to envision the effects of special loading conditions.

ACKNOWLEDGMENTS

Beams S1 through S4 were tested by A. V. Saracoglu as part of his master's degree program. The remainder of this study was sponsored by the Bureau of Engineering Research at the University of Texas.

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For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

Title No. 53-9

Conduction Calorimeter for Measuring Heat of Hydration of Portland Cement at Elevated Temperatures and Pressures*

By N. C. LUDWIG† and S. A. PENCE‡

SYNOPSIS

An apparatus for measuring continuously the rates and total amounts of heat liberated when cement compositions and water react at temperatures in the range of 80 to 350 F and at pressures from atmospheric to 10,000 psi is described. The method of calibration and test procedure are given.

Typical test results obtained on portland cement pastes cured at 100 F and at various pressures show that the rates and total amounts of heat liberated are increased by pressure at early ages.

INTRODUCTION

This paper describes an apparatus for measuring the quantity of heat liberated at elevated temperatures and pressures from portland cement pastes during the reaction of cement and water, except that released during the initial mixing of the slurry. The calorimeter unit is patterned after one developed by Carlson[1,2] and refined by Lerch.[3] The slurry cell, constant temperature cabinet, high-pressure pump, recording millivoltmeter and auxiliary equipment are new to this type of measurement.

In the placing of cement in deep oil wells[4] and in the steam curing of cement products, high temperatures and high pressures are involved. This apparatus, designed to operate at temperatures in the range of 80 to 350 F and at pressures from atmospheric to 10,000 psi, provides a means of studying portland cement pastes for these uses.

Siliceous materials are used in mixtures with cement to a considerable extent in the above and other applications. Most of these materials are not completely dissolved in calorimeter acid in the heat of solution method, ASTM C186-47,[5] normally used to determine the heat of hydration of portland cement. Hence,

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reliable results for heats of hydration cannot be obtained by this method on some of these blends. Also, the ASTM method is not suitable for measuring the rates at which heat is released from a cement paste continuously over a period of time.

The apparatus described provides a means for measuring the rate at which heat is liberated at any given period and, accordingly, the total amount of heat liberated. It also provides a method of measuring the reactivity in cement pastes of siliceous materials at different temperatures.

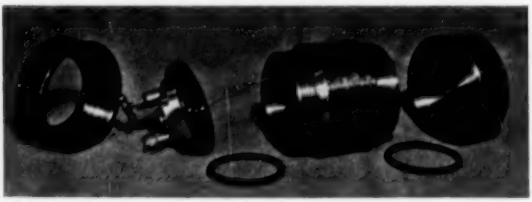


Fig. 1—Specimen cell disassembled

APPARATUS

The conduction calorimeter consists of a stainless steel slurry cell, a calorimeter unit, a liquid bath, and an insulated constant temperature cabinet. The disassembled specimen cell is shown in Fig. 1. This consists of:

1. The cell body, which is 3¾ in. high with an external diameter of 3 5/16 in. The inside diameters are 2¼ in. at the top and 2 in. at the bottom. The cell is tapered to permit the hardened paste to be removed at the end of the test. The volume of the space occupied by the cement slurry is 209.2 ml.
2. A top closure disc fitted with two high-pressure valves and connectors for an iron-constantan thermocouple which will be referred to as the cell-thermocouple. This disc is held in place by a threaded closure ring.
3. A one-piece bottom closure cap.
4. Two neoprene "O" rings which fit into grooves in the top and bottom of the cell body.

The principle upon which the apparatus operates is that of conducting the heat released, as the reactions occur, from the slurry cell through a calibrated conduction tube to the liquid in a constant temperature bath. The heat is removed, measured, and dissipated by a heat-conduction assembly. This consists of:

1. A 3 5/16 x ¼-in. circular copper pedestal-plate which supports the slurry cell.
2. A 5-in brass conduction tube having an internal diameter of 1 in. and outside diameters of 1⅛ in. for the top 3¼ in. and of 1¼ in. for the lower 1¾ in. of its length. Two brass thermocouple rings 2⅛ in. O.D. x 1⅛ in. I.D. x ¼ in. thick fit over the conduction tube. One is in thermal contact with the top pedestal-plate and the other is in thermal contact with the dissipator plate. These rings are grooved on the inside and drilled radially to permit the insertion of two diametrically placed copper-constantan thermocouples. These thermocouples are connected to form two sets of differential couples in series.
3. A 6½ x ¼-in. circular brass heat-dissipator plate, which has eight 2¼ x 1½ x ⅛-in. copper vanes silver soldered to the bottom face at 45-deg intervals. The vanes are also silver soldered to the lower 1¾ in. of the conduction tube.

4. Two laminated Bakelite rings, which are placed on the top of the heat-dissipator plate to support a Dewar flask shown in Fig. 2A.

This apparatus is assembled as shown in Fig. 2B. In use, it is supported by a short pedestal in the center of a liquid temperature bath and housed in a constant temperature cabinet. This assembly is shown in Fig. 3.

The liquid temperature bath consists of a stainless steel container fitted with an immersion heater and a stirrer driven with compressed air. Water covered with a thin layer of oil is used in the bath at temperatures up to 200 F. Either a vegetable oil or silicone is used at higher temperatures. The level of liquid in this bath is maintained slightly below the top surface of the heat-dissipator plate.

The cabinet is a double-walled steel box with sides, bottom and lid insulated with 4 in. of glass wool. It is equipped with two 500-watt screw socket type heating elements, an externally driven electric fan, and a mercury-in-glass thermostat. The thermostat is connected to an electronic relay which energizes a relay in the heating circuit. Fig. 4 shows the exterior of the cabinet, the control panel, and the high-pressure pump used to maintain pressure on the cement slurry.

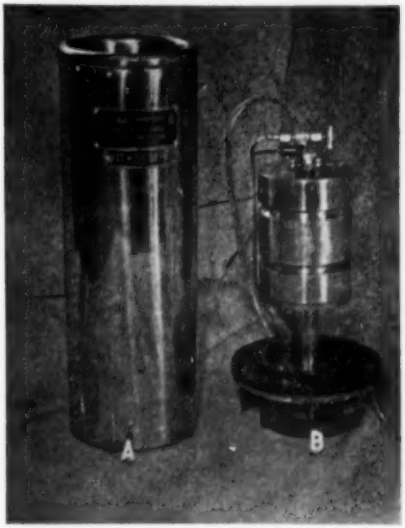


Fig. 2—(A) Stainless steel Dewar flask. (B) Cell and calorimeter

High-pressure steel tubing, thermocouple leads, thermostat leads, heater wires, and the air supply for the stirrer are brought into the cabinet through metal tubes in the side walls. The high-pressure steel tubing passes through a hole in the heat-dissipator plate and along the side of the slurry cell to one of the valves on the top closure disc of the cell. The leads for the iron-constantan cell thermocouple pass through a small radial hole in the Bakelite ring, on top of the heat-dissipator plate, and go to the thermocouple connector plug on top of the cell.

Two copper-constantan thermocouples are used to measure the bath temperature and the air temperature, respectively. One is tied onto one of the vanes of the heat-dissipator and the other is tied to the outside of the Dewar flask.

A special Leeds and Northrup, double-range, four-point recording millivolt meter, shown in Fig. 4 on the lower left of the panel board, measures and records the thermocouple voltages. There is one point on the "low range" of −0.10 to +0.90 mv. This is connected to the conduction tube differential couple to measure the heat flow. The remaining three points are on the "high range" of 0 to 10 mv and are connected to the cell, bath, and air thermocouples. As this instrument does not contain a junction temperature compensator, an ice junction for these three

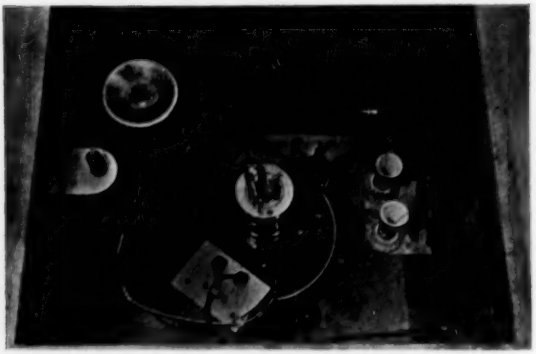


Fig. 3—Arrangement of apparatus in cabinet. In operation the Dewar flask sets over cell and calorimeter

couples is located on a shelf in back of the instrument.

An air-operated, 0 to 30,000 psi capacity Sprague pump, which uses either water or oil as a pressure medium, is located back of a small pressure cell as shown in Fig. 4.

METHOD OF TEST

Calibration of apparatus

The calibration consists of two parts as follows:

1. Heat of conduction, *i.e.*, determining the relationship between the rate at which heat is removed from the specimen cell and the emf developed by the differential thermocouple.
2. Heat of storage, *i.e.*, determining the relationship between the amount of heat stored in the specimen and specimen cell and the emf developed by the differential thermocouple.

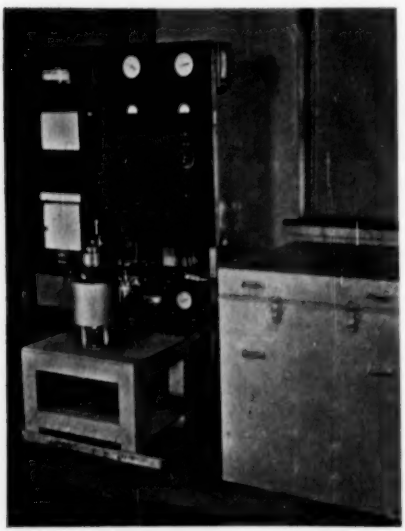
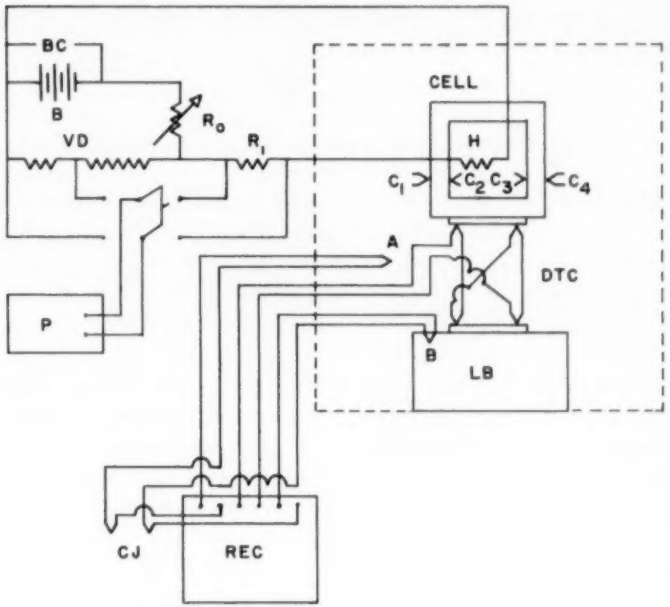


Fig. 4—Constant temperature cabinet, pressure pump, control panel, and curing cell

The first calibration was made by measuring the emf produced by the differential thermocouple by heating the specimen cell with a manganin resistance heater placed inside the cell. This heater was connected through an adjustable resistance to a 6-v storage battery. Different constant amounts of current were supplied to give different rates of heat input to the cell. The apparatus was operated at each rate until a steady state was obtained before data used in the calibration were taken. The



B - 6 VOLT STORAGE BATTERY
BC - BATTERY CHARGER TO STABILIZE VOLTAGE
R_0 - VARIABLE RESISTANCE FOR HEAT INPUT ADJUSTMENT
VD - VOLTAGE DIVIDER, RATIO 1:100
R_1 - STANDARD RESISTANCE, 0.10 OHM
P - PRECISION POTENTIOMETER
H - MANGANIN HEATER COIL
C_1, C_2, C_3, C_4 - OUTSIDE AND INSIDE CELL THERMOCOUPLES USED FOR TEMPERATURE CALIBRATION. THESE ARE CONNECTED THROUGH AN ICE JUNCTION AND SWITCHES TO POTENTIOMETER P.
DTC - DIFFERENTIAL THERMOCOUPLES
A - CABINET AIR TEMPERATURE THERMOCOUPLE
B - LIQUID BATH TEMPERATURE THERMOCOUPLE
LB - LIQUID BATH
CJ - COLD JUNCTIONS FOR A AND B THERMOCOUPLES
REC - L&N DOUBLE RANGE MILLIVOLT RECORDER

Fig. 5—Calibration circuit and list of apparatus

measuring circuit and a list of the apparatus used in both calibrations are shown in Fig. 5. The average of ten sets of readings of volts, amperes, and differential thermocouple emf were used to calculate the heat of conduction values.

The rate at which heat was supplied and conducted from the cell may be calculated by the formula:

$$H_{ct} = 860EI \quad \ldots\ldots (1)$$

where

H_{ct} = total rate of heat conduction, cal per hr
E = equilibrium voltage across the heating coil
I = equilibrium current through heating coil
860 = cal in one watt-hr

The heat of conduction per g of cement released by the cement slurry in the cell may be calculated by the formula:

$$H_{cc} = \frac{860EI}{W} \quad \text{.....(2)}$$

where

H_{cc} = unit rate of heat conduction, cal per g of cement per hr
W = g of cement in the cell

The calibration data obtained at 100 F are given in Table 1a. Heat of conduction values are expressed both in cal per hr and cal per g of cement per hr; the la ter values are for a slurry containing 100 parts cement and 46 parts water. A plot of differential thermocouple emf values against 860 *EI*, as shown in Fig. 6, gave a straight line. This was used to obtain a constant for converting the emf readings to unit heat of conduction values.

The second calibration was made by measuring the average temperature of the slurry cell, the bath temperature, and the differential thermocouple emf when a constant voltage was applied to the manganin heater. The arrangement of thermocouples used to measure the temperature of the cell is shown in Fig. 5. These measurements were made at 30-min intervals over the period required for the emf of the differential thermocouple to indicate a temperature difference between the top and bottom of the conduction tube of 30 F.

TABLE 1—CALIBRATION DATA FOR CONDUCTION CALORIMETER AT 100 F

a—Heat of conduction

Volts	Amperes	Watts	Differential couple emf, mv	Total rate of heat conduction, cal per hr*	Unit rate of heat conduction†, cal per g cement per hr
0.0	0.0	0.0	0.01‡	0.0	0.0
0.5931	0.1198	0.0710	0.0261	61.060	0.227
0.9998	0.2017	0.2016	0.0527	173.37	0.644
2.071	0.4178	0.8653	0.1755	744.16	2.766
3.361	0.6788	2.281	0.4440	1962.0	7.291

b—Heat of storage

Difference between cell and bath temperature		Differential couple emf	Heat of storage†	
deg F	deg C	mv§	cal	cal per g cmt
Heating cycle				
11.4	6.33	0.482	5776.0	21.46
21.7	12.06	0.908	11004.5	40.89
28.9	16.06	1.205	14654.4	54.46
38.5	21.39	1.620	19518.0	72.54
Cooling cycle				
28.4	15.78	1.158	14398.9	53.51
20.4	11.33	0.830	10338.4	38.42
16.9	9.39	0.692	8568.2	31.84

*The equation for the relationship between the total rate of heat conduction and the emf difference is $H_{cc} = 4545$ (emf—0.01‡).

†Calculated on the basis that the specimen cell contains 209.2 ml of slurry consisting of 100 parts cement and 46 parts water.

‡Error in zero reading of recorder—since corrected.

§The equation for the relationship between the heat of storage and the emf difference is $H_{cc} = 12297$ ±emf).

From these data, the amount that the average temperature of the cell exceeded the temperature of the bath was calculated. Readings were also taken when the specimen cell was cooling to determine whether or not the values obtained at decreasing rates of heat storage were the same as those obtained at increasing rates. The values were found to be approximately the same.

The heat of storage for each differential thermocouple reading was calculated by the equation:

$$H_{st} = C(T_s - T_b) \quad \ldots\ldots (3)$$

where

H_{st} = total heat stored, cal

C = heat capacity* of specimen cell, top pedestal-plate, and cement slurry calculated from the weights and specific heats of these components

T_s = average temperature of the specimen cell, deg C

T_b = temperature of liquid bath, deg C

The calibration data obtained for heat storage at 100 F are given in Table 1b. These values are expressed both in terms of total heat stored, cal, and unit heat

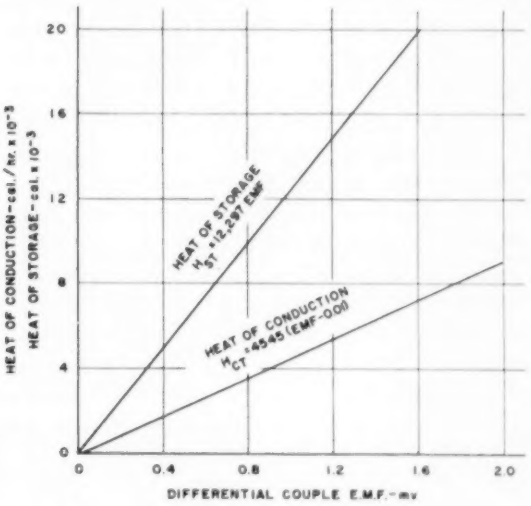


Fig. 6—Relationship between total rates of heat conduction, heat of storage, and emf difference at 100 F

stored, cal per g of cement. Fig. 6 shows a plot of differential thermocouple emf values versus the total heat of storage as obtained by Eq. (3). This was used to obtain a constant for converting the emf reading to unit heat of storage values.

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* C, cal per deg C = $(W_{st})(S_{st}) + (W_p)(S_p) + (W_c)(S_c) + (W_w)(S_w)$ where W_{st}, W_p, W_c, and W_w are the weights in g of the stainless steel cell, copper pedestal, cement, and water, respectively, and S_{st}, S_p, S_c, and S_w are the specific heats of these materials.

For this system with a slurry containing 100 parts cement and 46 parts water, the numerical values are C, cal per deg C = (5793) (0.120) + (422) (0.0939) + (269.08) (0.20) + (123.78) (1.0008) = 912.48.

Small differences in the amounts of slurry placed in the cell do not significantly change the total heat capacity of this system. Therefore, the value of C in Eq. (3) was not corrected when the weight of cement + water differed slightly from 392.86 g.

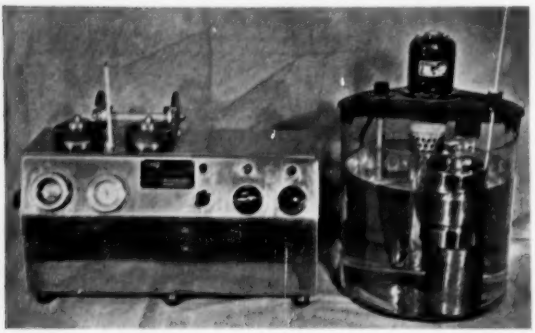

Fig. 7—Halliburton tester and constant temperature bath

Sample preparation

The following procedure* was found suitable for test temperature up to 200 F. When a test is started, the temperature of the slurry cell and contents should be as nearly equal to the cabinet and bath temperature as possible. To satisfy this condition, the dry cement, water, and specimen cell are preheated overnight to test temperature in the calorimeter cabinet. Just before starting a test, the specimen cell is transferred to a constant temperature bath also at test temperature. Mixing of the preheated cement and water is done in one cell of a Halliburton thickening-time tester[6] at test temperature. These pieces of apparatus are shown in Fig. 7.

The mixing schedule is as follows: (1) add the cement to water in the Halliburton cell and start the recorder; (2) mix slurry by hand 2 min with a 1-in. spatula; (3) place cell in Halliburton tester and mix 3 min; (4) remove cell and weigh; (5) pour slurry into specimen cell to mark; (6) weigh Halliburton cell again and calculate amount of slurry in specimen cell by difference; (7) close specimen cell, place back in temperature bath, and fill with water through valves; (8) remove specimen cell from bath at end of 15 min from the time cement was added to water and place on calorimeter pedestal; (9) connect cell-thermocouple and pressure line; (10) cover assembly with Dewar flask; (11) close cabinet; and (12) remove short from differential thermocouple lead. By careful adjustment of temperature of the water bath used in step 7, it is possible to start a test with a differential thermocouple reading of 0.0 $\pm$ 0.02 mv.

Calculation of heat of reaction

The heat of conduction at each hour out to 72 hr was calculated by multiplying the corrected differential couple emf readings (mv $-$ 0.01) by the conduction constant, 4545, and dividing by the weight of cement in the cell. The heat of storage $+$ or $-$ was obtained by calculating the change in mv readings from one

*Some preliminary tests were made at 300 F with slow-set cement. In these tests, the procedures outlined above were carried out at 200 F. The closed cell and contents were placed in the curing vessel, shown in Fig. 4, and heated rapidly to 300+ F and then placed on calorimeter pedestal.

TABLE 2—TYPICAL TEST RESULTS AND CALCULATION OF RATES AND TOTAL HEATS OF HYDRATION FOR AN ASTM TYPE I CEMENT CURED AT 100 F, ATMOSPHERIC PRESSURE*

Time, hr	Differential couple emf, mv	Differential couple,† emf change, mv	Heat of conduction,‡ cal per g per time interval indicated	Heat of storage,§ cal per g per time interval indicated	Rate of hydration,¶ cal per g per hr	Total heat of hydration,** cal per g
0	0.000	0.000	0.000	0.000	—	—
0.5	0.000	+0.003	0.000	+0.137	—	—
1	0.004	+0.005	0.000	+0.228	0.365	0.365
1.5	0.015	+0.016	0.042	+0.732	—	—
2	0.034	+0.021	0.203	+0.960	1.937	2.302
2.5	0.052	+0.016	0.355	+0.732	—	—
3	0.070	+0.018	0.507	+0.823	2.417	4.719
3.5	0.090	+0.024	0.676	+1.097	—	—
4	0.118	+0.020	0.912	+0.914	3.599	8.318
4.5	0.147	+0.028	1.156	+1.280	—	—
5	0.173	+0.023	1.377	+1.050	4.863	13.181
5.5	0.194	+0.018	1.554	+0.823	—	—
6	0.210	+0.017	1.689	+0.777	4.843	18.024
6.5	0.220	+0.010	1.774	+0.457	—	—
7	0.228	+0.007	1.842	+0.320	4.393	22.417
7.5	0.233	+0.002	1.884	+0.091	—	—
8	0.233	−0.002	1.884	−0.091	3.768	26.185
8.5	0.231	−0.002	1.866	−0.091	—	—
9	0.227	−0.005	1.832	−0.228	3.379	29.564
9.5	0.221	−0.007	1.782	−0.320	—	—
10	0.213	−0.008	1.715	−0.365	2.812	32.376
10.5	0.207	−0.008	1.664	−0.365	—	—
11	0.200	−0.007	1.606	−0.320	2.585	34.961
11.5	0.190	−0.009	1.520	−0.412	—	—
12	0.183	−0.006	1.461	−0.274	2.295	37.256
13	0.168	−0.009	2.669	−0.412	2.257	39.513
14	0.155	−0.014	2.450	−0.640	1.810	41.323
15	0.141	−0.013	2.213	−0.594	1.619	42.942
16	0.129	−0.013	2.011	−0.594	1.417	44.359

*W/C = 0.46; weight of cement in cell = 269.08 g.

†Change in differential couple mv readings 15 min before to 15 min after time indicated for 0.5-hr interval calculations. In the case of 1-hr interval calculations made after 12 hr, which is the change from 30 min before to 30 min after time indicated.

‡Conduction constant, cal per g of cement per hr per mv = 4545/269.08 = 16.89. (Column 2 values −0.01) mv × 16.89/2 for 0.5-hr intervals or (Column 2 values −0.01) mv × 16.89 for 1-hr intervals.

§Storage constant, cal per g of cement per mv = 12,297/269.08 = 45.7. Column 3 values × 45.7.

¶Alegbraic sum of Column 4 and Column 5 values.

**Subtotals of Column 6.

period to the next and multiplying this by the heat of storage constant, 12,297, and dividing by the weight of cement in the cell. These constants are shown in Tables 1a and 1b, and more details concerning these calculations are given at the end of Table 2. The algebraic sum of unit rate of heat of conduction and unit rate of heat of storage is the rate of heat liberation at any particular hour. The sum of these hourly rates is the heat of hydration for a given period. During the first 12 hr, when the reaction is proceeding at a relatively rapid rate, heat of conduction and heat of storage values were calculated at 30-min intervals.

A typical set of data and the calculation of the heat of hydration are shown in Table 2.

TEST RESULTS

The first series of tests made with this calorimeter were run in duplicate on an

TABLE 3—RESULTS OF HEAT OF HYDRATION TESTS ON ASTM TYPE I CEMENT CURED AT 100 F AND DIFFERENT PRESSURES*

Heat of hydration at time indicated, cal per g of cement Conduction calorimeter										ASTM C 186, method 72 hr
1 hr	2 hr	4 hr	6 hr	8 hr	12 hr	18 hr	24 hr	48 hr	72 hr	
Curing pressure—Atmospheric										
0.6	2.5	10.2	19.6	27.5	37.9	45.8	51.6	60.1	63.6	62.7
0.4	2.3	8.3	18.0	26.2	37.3	46.8	52.1	60.8	63.7	
0.5	2.4	9.2	18.8	26.8	37.6	46.3	51.8	60.4	63.6	
Curing pressure—1000 psi										
0.2	2.1	10.5	20.5	28.5	39.0	47.8	53.0	62.4	66.2	66.6
0.4	2.0	11.3	21.3	29.2	39.6	48.6	53.1	61.8	65.4	
0.3	2.0	10.9	20.9	28.8	39.3	48.2	53.0	62.1	65.8	
Curing pressure—5000 psi										
0.7	3.4	16.6	27.4	35.4	43.4	50.9	55.0	62.2	65.6	68.2
0.9	4.3	15.9	26.8	34.4	44.1	51.3	55.4	62.6	66.1	
0.8	3.8	16.2	27.1	34.9	43.8	51.1	55.2	62.4	65.8	
Curing pressure—7500 psi										
1.8	6.3	20.2	30.6	37.2	45.5	51.2	55.6	63.0	66.8	68.3
1.4	6.3	20.3	31.0	38.0	46.8	53.6	57.1	65.2	69.2	
1.6	6.3	20.2	30.8	37.6	46.1	52.4	56.3	64.1	68.0	

*$W/C = 0.46$.

ASTM Type I cement at 100 F* at pressures of 0, 1000, 5000, 7500 psi and at a W/C of 0.46 by weight. At the end of 72 hr, the sample was removed from the calorimeter and a portion of it was used to determine the heat of hydration by the heat of solution method (ASTM C 186-47). The test results are shown in Table 3 and the rates of hydration and total heats of hydration calculated from the conduction calorimeter data are plotted in Fig. 8.

The greatest differences between the values obtained in the conduction calorimeter and those obtained by the heat of solution method at 72 hr were obtained in tests at 5000 psi. These differences were $68.2 - 65.8 = 2.4$ cal per g, using the average value; and $68.2 - 65.6 = 2.6$ cal per g, using the lowest result. A part of these differences is due to the fact that values by the heat of solution method include the heat released during the initial mixing of the slurry.

With reference to Fig. 8, note that pressure accelerated the rate of hydration of this cement at early ages, especially up to about 6 hr. After final set, at about 8 hr, the pressure did not have much effect. It is beyond the scope of this paper to offer an explanation for these effects. However, it is planned to present more data and explanations in future papers.

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*Actually, the cement reached a maximum temperature 6 to 8 F above bath temperature.

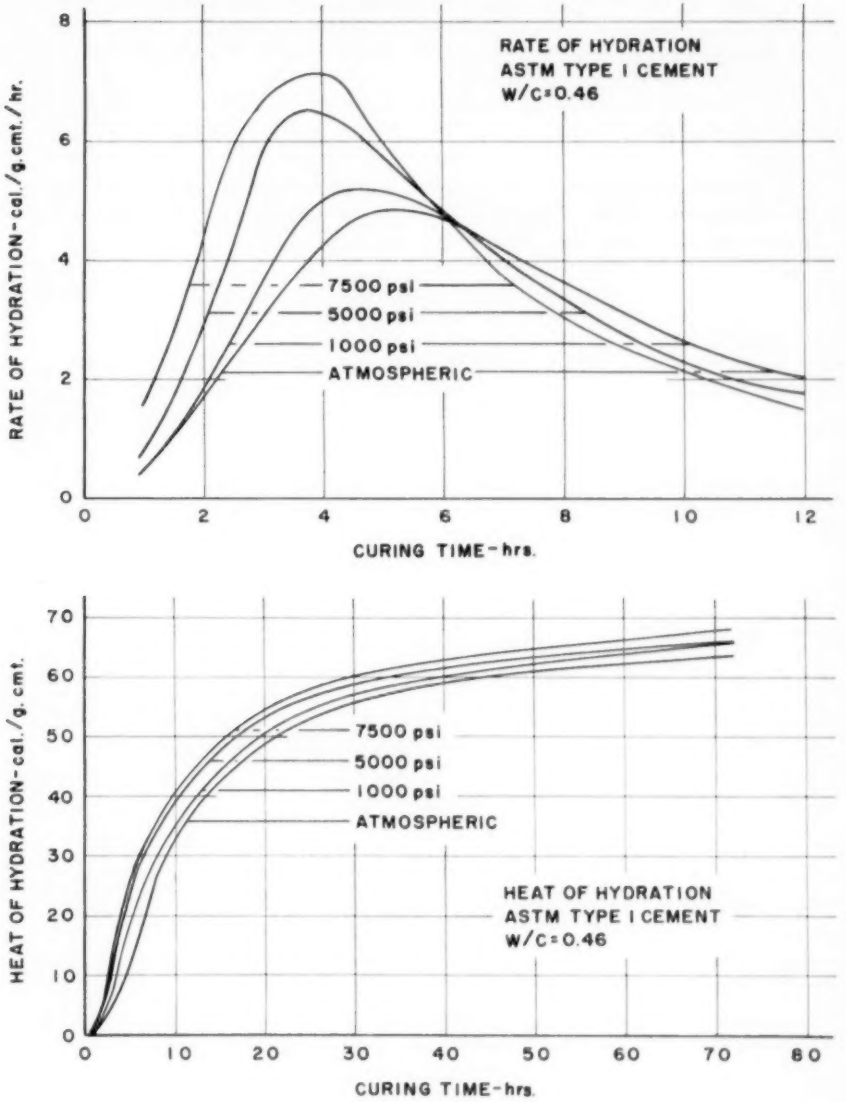

Fig. 8—Effect of pressure on heat of hydration at 100 F

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For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

Title No. 53-10

Bhakra Dam—Design and Construction Features*

By C. L. HANDA† and O. P. CHADHA‡

SYNOPSIS

The Bhakra Nangal project is one of the multipurpose projects being constructed in India for irrigation and power. Major feature of the project is Bhakra Dam, a 680 ft high gravity concrete structure. General design features of Bhakra Dam are described including geology of the dam site, layouts, galleries, outlets and spillway, and foundations.

In the section on construction of the dam, aggregate cooling, temperature control of the concrete to minimize cracking, mix proportioning, plant layout, and concreting set-up are described. Considerable attention is given to the raw materials, particularly the aggregate and pozzolans, and another section covers various concrete investigations.

INTRODUCTION

The Bhakra Nangal project is one of India's major multipurpose irrigation and power projects. It envisages construction of Bhakra Dam, a 680 ft high straight gravity concrete structure; an auxiliary diversion dam, Nangal Dam; the Nangal-Hydel Canal with its two power plants; and the Bhakra canals and their distributory systems.

Full scale construction started in 1948 and is expected to culminate in 1960 with completion of Bhakra Dam and power plant. The dam will store about 7,500,000 acre-ft of water and generate over 400,000 kw of firm power. When completed, the $350,000,000 project will irrigate 6,000,000 acres. The irrigation system comprises some 3000 miles of channels fed from three river diversion dams acting in conjunction with Bhakra Dam. Nangal Dam and Nangal-Hydel Canal (Fig. 1)—a lined channel of 12,500 cfs discharge—were opened in July, 1954, followed by Ganguwal power plant on the canal in January, 1955. The second power plant, Kotla, started operation in mid-1956. Each of these two power plants will have an ultimate installed capacity of 72,000 kva each.

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Fig. 1—Nangal Hydel Canal bypass near Ganguwal power house

DESIGN FEATURES OF DAM

Foundation problems

Bhakra Dam is in Punjab State about 200 miles northwest of New Delhi where the Sutlej River cuts a narrow gorge through the last range of the Himalayas. The foundations and abutments of the dam site are preponderantly of sandstone with important claystone and siltstone layers intervening between the sandstone horizons. The stratification is about at right angles to the gorge with beds tilted at 70 deg downstream, but this tilt is not uniform. At higher elevations on the abutments there is flattening of inclination, and at lower elevations the formation tends to become vertical.

The complexity of the problem is intensified by existence of shear and thrust zones in otherwise sound rock, by jointing both along and across the bedding planes, by weathering effects, and by occurrence of weaker claystone-siltstone bands. The two major bands are called the upstream claystone stratum and the middle third claystone stratum. The upstream claystone stratum is the principal foundation fault and consists of a 100 ft wide band just 75 ft upstream from the dam axis.

The presence of claystone between sandstone members created special problems. The claystones are in themselves compact and dense, but there is evidence of shear

zones and minor cracks, which may lead to deterioration if ingress of water occurs. To guard against such deterioration, the construction program calls for chemical and fine cement grouting of the enclosing sandstone members, and covering the claystone stratum itself with concrete. A plug of concrete not less than 75 ft thick is proposed to be laid over the freshly exposed claystone in the trench dug into the heel claystone member. A network of galleries is being left to gain access to the foundations for initial and post-reservoir grouting.

Central structure

The dam is a straight gravity concrete structure, rising 680 ft above the lowest foundations with a crest length of 1617 ft.

In the abutment section the dam has a slope of 0.8 to 1 from the lowest point at the downstream edge of the base to el 1662.5 with P.I. at el 1700. Above this elevation the dam is 30 ft thick with downstream face vertical. The upstream face of the dam has a slope of 0.35 to 1 from the base to el 1350, above which point the face becomes vertical.

Two towers, one on either side of the spillway, surmount the crest of the dam to house access elevators.

A large number of instruments are to be embedded in the dam to study its structural behavior. These instruments will measure temperatures within the mass of the dam, openings of the longitudinal and transverse contraction joints, stresses and strains in concrete, and uplift pressures on the base profile of the dam. In addition, three plumb lines extending to almost full height of the cantilevers have been included in the design to measure deflections of the dam.

The dam rests upon the largest member of the sandstone series. Upstream and downstream from the dam there are wide claystone bands which closely limit the location of the dam. The gorge is narrow with comparatively steep rock walls. While this indicates an economical dam site, it increases the problem of finding space for all the appurtenant works like the power plant, spillways, and irrigation outlets.

Spillway

Discharge observations of Sutlej River at Bhakra dam site have been made since 1909. A flood of 354,000 cfs has been computed to have a frequency of once in 1000 years but the design flood has been taken to be 400,000 cfs having a frequency of once in 10,000 years.

Discharge of flood waters over the 260 ft wide spillway near the center of the dam (Fig. 2) will be controlled by four 50 x 37.5-ft radial gates. This spillway extends down to form a concrete apron, as just downstream of the theoretical toe of the dam, there is a 250 ft wide claystone band which must be protected against action of water for structural reasons.

The discharge intensity per foot run of the spillway is 1160 cfs; the corresponding figures for Shasta and Grand Coulee dams are 900 and 894, respectively. Final shapes of the crest and the apron were arrived at after extensive model studies.

Besides two side training walls, a central dividing wall extends from the crest to the end of the spillway apron. This will allow closing half of the spillway for periodic inspection and repairs.

Galleries and tunnels

For access to the body of the dam and the foundations, 14,500 ft of galleries have been provided. The galleries are rectangular in section, 5 ft wide by 7 ft high. Some longitudinal foundation galleries were made 6 x 7 ft to facilitate drilling and grouting operations.

Twelve grouting and drainage tunnels, 6 x 7 ft finished size, are being driven into the hillsides at various elevations. These concrete lined tunnels are generally parallel to the axis of the dam, and will provide access for consolidation of the abutment rock, thereby reducing seepage from the reservoir. The foundation drainage from tunnels at lower elevations will prevent uplift pressures building up against the right and left power plants when the reservoir fills, since the power plants are near the downstream toe of the dam.

River outlets

Ten 15-ft diameter penstocks and twenty 8 ft 6 in. diameter river outlets will release water for irrigation and power. The river outlets, in the spillway section of the dam (Fig. 2) are arranged in two tiers of ten each at el 1320 and 1420. At the full reservoir of el 1680 the river outlets have a discharge capacity of 61,000 cfs for the lower and 54,000 cfs for the upper tiers. Together these outlets can carry up to 40 percent of the total spilling capacity of the dam. Discharge will be regulated by 20 jet-flow fixed wheel gates on which the maximum head is more than 368 ft.

CONSTRUCTION OF DAM

The gigantic task of building the highest straight gravity dam in the world is being done by the Punjab government departmentally, with the help of foreign specialists, most of whom are from the United States. Modern construction methods and the best of modern plant equipment are being employed for speedy and efficient completion of the project (Fig. 3).

As of June, 1955, about 70 percent of the total of more than 5,000,000 cu yd of excavation was completed. Both upstream and downstream coffer dams 200 ft and 128 ft above the foundations have been completed. The river is now flowing through two 50-ft diameter concrete lined tunnels, each about ½ mile long (Fig. 4). The construction plant will be prepared to start placing concrete by the end of this year, and will then continue, practically ceaselessly day and night, for the next 3½ years. To obtain 5,000,000 cu yd of concrete required for the dam and appurtenant works, some 8,800,000 tons of processed aggregate and 700,000 tons of cement are required.

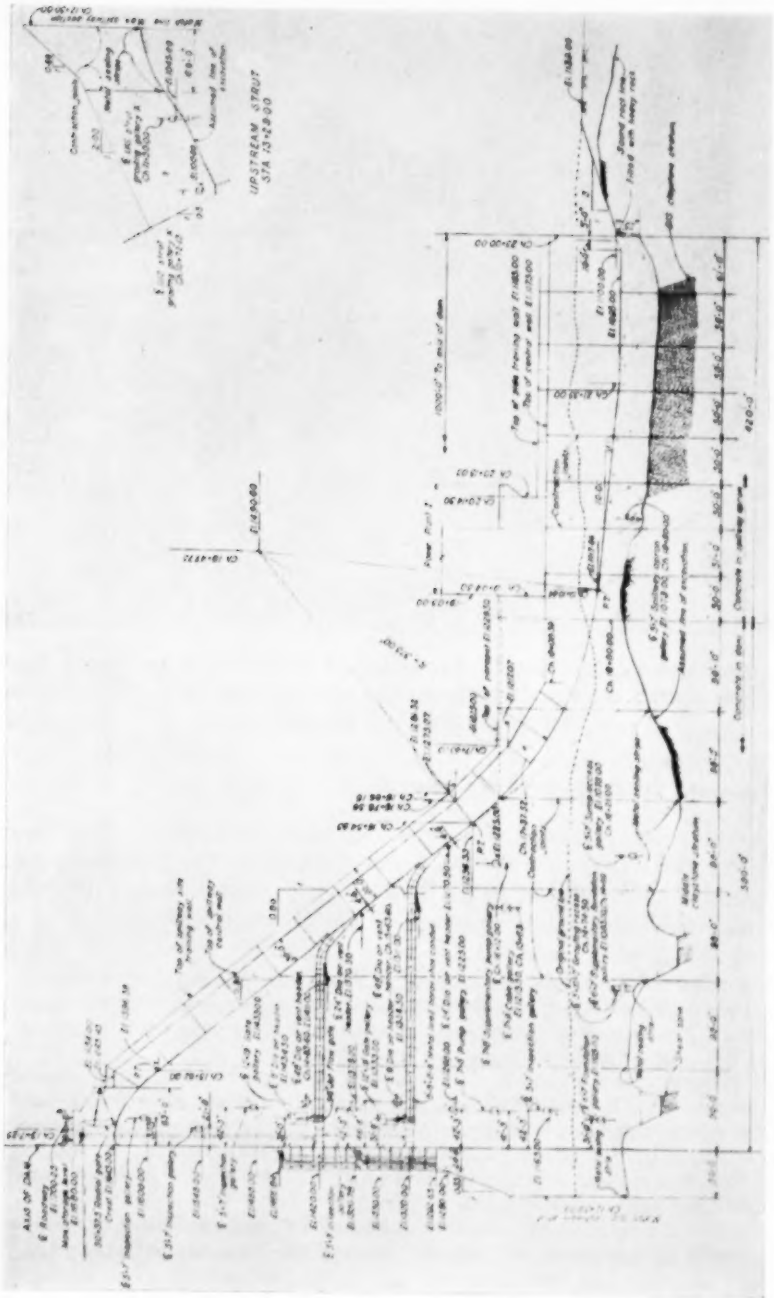

Fig. 2—Maximum spillway section at Station 13 + 29.00

Fig. 3—Bhakra construction plant area. Offices, field workshops and screen house are at upper center; cooling plant upper right; cement silos right center; cement unloading shed at lower right

Mass concrete cracking and its control

Due to the dimensions of Bhakra Dam, one of the most important problems is the selection of an economical scheme of concrete placement that allows maximum control of temperature in all stages of concreting, both to reduce the formation of undesirable shrinkage cracks and to enable scheduled placement of an average of 5000 cu yd of concrete daily.

After a number of studies, the scheme accepted has these salient features:

(a) The dam is divided into blocks or columns by transverse and longitudinal joints. The transverse joints are spaced 50-60 ft apart, the usual length of blocks in the other direction being 100 ft.

(b) The concrete is to be placed in 6-ft lifts as against the standard American practice of 5-ft lifts. This change is expected to speed construction and effect a saving in clean-up of lift tops. A minimum interval of 72 hr will be allowed before the top of a lift can be covered with fresh concrete.

(c) Standard cement (corresponding to ASTM Type I) with as low a content per cubic yard as possible will be used.

(d) The concrete shall be precooled to about 65 F when deposited in forms. Final stable temperatures of a large portion of the dam are expected to be 65 F.

(e) Cooling with an embedded system of thin wall tubing to final stable

Fig. 4—Bhakra right diversion tunnel, 50-ft finished diameter, seen at inlet end

temperatures will be done just prior to grouting of the contraction joints.

This scheme is flexible, offers positive control of temperature, and will enable concrete to be placed year-round without interruption.

Aggregate handling

Aggregate sources are river gravel and sand deposits at Neilla Terrace and Fatehwal *khad* about 3 miles downstream from the dam site. The Fatehwal aggregate is deficient in coarse aggregate fractions from 1½ to 7 in. and that from Neilla deficient in fines. Therefore, it was decided to use about 15 percent of the aggregate from Neilla with 85 percent from Fatehwal. It has been estimated that roughly 6,000,000 cu yd of material will be required from Fatehwal and 1,500,000 cu yd from Neilla Terrace.

Based on the optimum output of the batching and mixing plant, the maximum quantity of processed aggregates required per month will be about 378,000 tons. For this about 584,000 tons of raw aggregate will have to be excavated. The project will require excavation of approximately 10,000,000 tons of aggregates at Fatehwal and 2,500,000 tons at Neilla.

Scalping units at Fatehwal and Neilla will reject sizes over 7 in. and ¾ in., respectively. The raw aggregates will then be transported to the 25,000-ton main surge pile at Bhakra processing plant by a 36-in. belt conveyor 9352 ft long (Fig. 5). Composed of seven flights, moving at 450 ft per min, it is capable of transporting over 900 tons per hr. All multiple flight conveyors are electrically interlocked. In case of stoppage or failure of any one flight, all the preceding flights of belt conveyors halt automatically. Telephones have been provided at all transfer points.

The processing plant at Bhakra (Fig. 6) will screen the aggregates, classify sand, and control crushing to make up any deficiencies. This will be achieved by various sizes of double-deck vibrating screens, Dorco rake classifiers and sizers, and cone crushers. Capacity of the plant is about 850 tons per hr.

Typical proportions of aggregate for mass concrete mix and approximate quan-

Fig. 5—Bridge across Sutlej River with belt conveyors being installed to bring aggregate from Neilla to Bhakra

tities required to be produced per hour are indicated in Table 1. The raw aggregate as it reaches Bhakra is well graded and does not require much crushing or wasting, the only wastage being in the form of unwanted finer fractions of sand.

Aggregate cooling

The specifications require that mass concrete in the dam shall be at a temperature of 65 F when deposited in forms. If artificial precooling of concrete were not resorted to, 85 F would be about the maximum reasonable temperature at which the concrete could be placed throughout the year without too many shutdowns due to high ambient temperatures.

TABLE 1—AGGREGATE FOR MASS CONCRETE

Size	Weight, lb	Tons per hr
7–3 in.	905	150
3–1½ in.	686	115
1½–¾ in.	604	100
¾ in.–No. 4	550	90
Sand	918	150
Total	3663	615

Part of the concrete, about 500,000 cu yd to be placed in foundation faults and spillway apron, is required to be specially precooled to a temperature of 50 to 55 F.

Cooling the coarse aggregate, about 410 tons per hr, to about 38 F and using chilled water for mixing are sufficient to attain a placing temperature of 65 F safely (Table 2). Total refrigeration capacity including provision for losses and standby is 1800 tons. The various coarse-aggregate fractions will be immersed in steel tanks (Fig. 7) through which chilled water will be circulated.

Cement handling and storage

Latest schedules indicate the maximum quantity of concrete to be placed per month is 193,000 cu yd requiring about 870 tons of cement daily. The cement will be manufactured by Indian mills conforming to IS 269-51, which is similar in

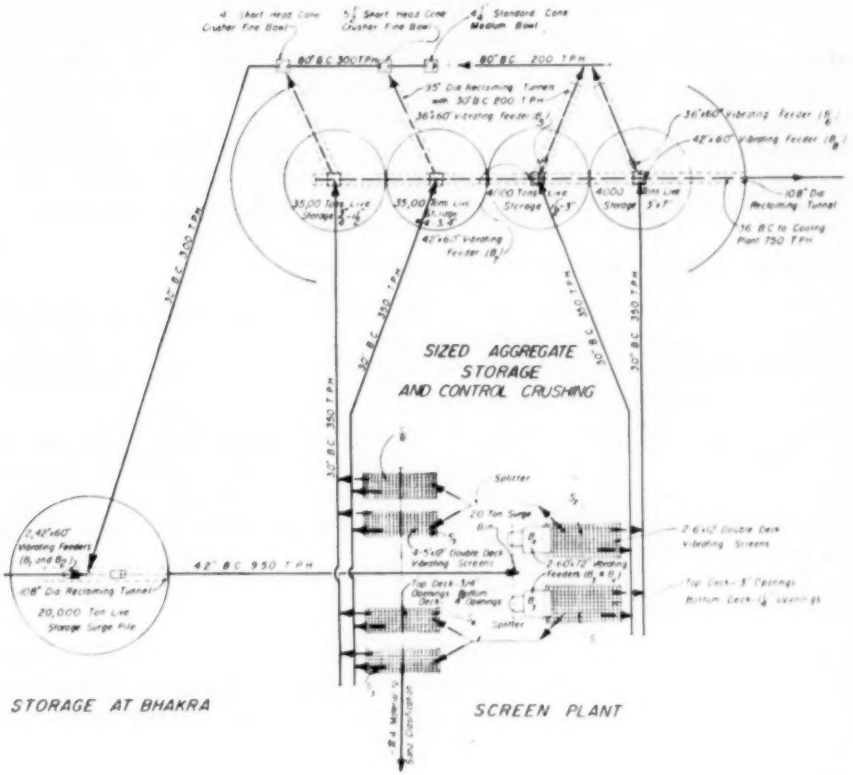

Fig. 6—Diagram of aggregate screening plant shows Bhakra storage at left. Aggregates are brought to live storage pile at Bhakra from Fatehwal and Neilla by belt conveyors

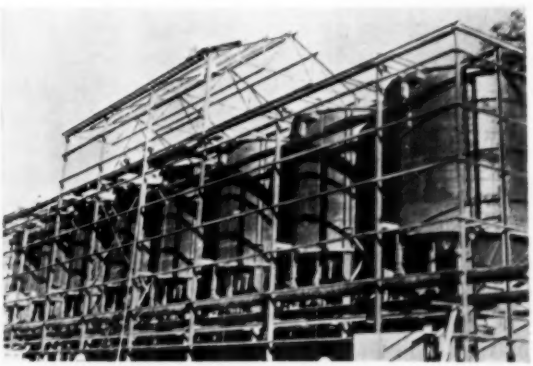

Fig. 7—Aggregate cooling tanks. Building to house them is under erection

thermal and other properties to ASTM Type I.

The 20-ton hopper bottom railroad cars bringing cement from the mills discharge into track hoppers at the railway siding near the dam site. Fuller-Kinyon stationary pumps, capable of pumping 100 tons of cement per hr, transfer the cement from the hoppers to any of the seven 840-ton steel storage silos at el 1300. From the silos the cement is pumped to a service silo at el 1500 by two pumps and from there to the batching and mixing plant at the first stage through Fuller-Kinyon Airslides and during the second stage by another pneumatic pump.

Batching, mixing, and concrete transportation

A batching and mixing plant with four 4-cu yd mixers, planned for an optimum output of 320 cu yd of concrete per hr, has been installed at el 1361.5 on the right side just downstream of the dam for the first stage. This plant will be shifted to el 1578.7 for concreting the second stage.

Concrete will be conveyed from the batching and mixing plant to the site in 4-cu yd bottom dump, air-operated buckets. Diesel electric cars, capable of carrying four loaded buckets with space for another empty bucket, will haul the concrete from below the concrete discharge hoppers to a point under the cranes where the buckets will be picked up and dumped.

Concrete placement

A two-stage system of trestles, with two double cantilever hammerhead cranes and three whirly cranes, have been selected for placement of concrete. These will be augmented by two stiffleg derricks and mobile cranes for placing concrete in zones not covered by the main system (Fig. 8).

TABLE 2—COMPUTATION OF CONCRETE PLACING TEMPERATURE

Material	Specific heat	Weight, lb	Weight of free moisture, lb	Water equivalent weight, lb	Input temperature, deg F	Total Btu
Cement	0.22	282	0	62.0	180	11,160
Sand	0.22	812	81	260 0	90	23,400
No. 4–¾ in.	0.22	516	8	121.7	38	4,600
¾–1 ½ in.	0.22	631	6	145.0	38	5,500
1 ½–3 in.	0.22	717	7	165.0	38	6,280
3–7 in.	0.22	1004	10	230.6	38	8,740
Water	1.00	58	—	58.0	35	2,030
Total				1042.3		61,710
(Mechanical heat of mixing = 1060 Btu)						1,060
						62,770

Temperature of the mixture = $\frac{62770}{1042.3}$ = 60 F

The double cantilevers have a total reach of 320 ft and are mounted on gantries more than 70 ft high. The whirlys are mounted on 60 ft high gantries and have an operating radius of 160 ft with a fully loaded 4-cu yd bucket.

There will be four trestles in the first stage for concreting up to el 1350 and one in the second stage of concreting above el 1350 to the top.

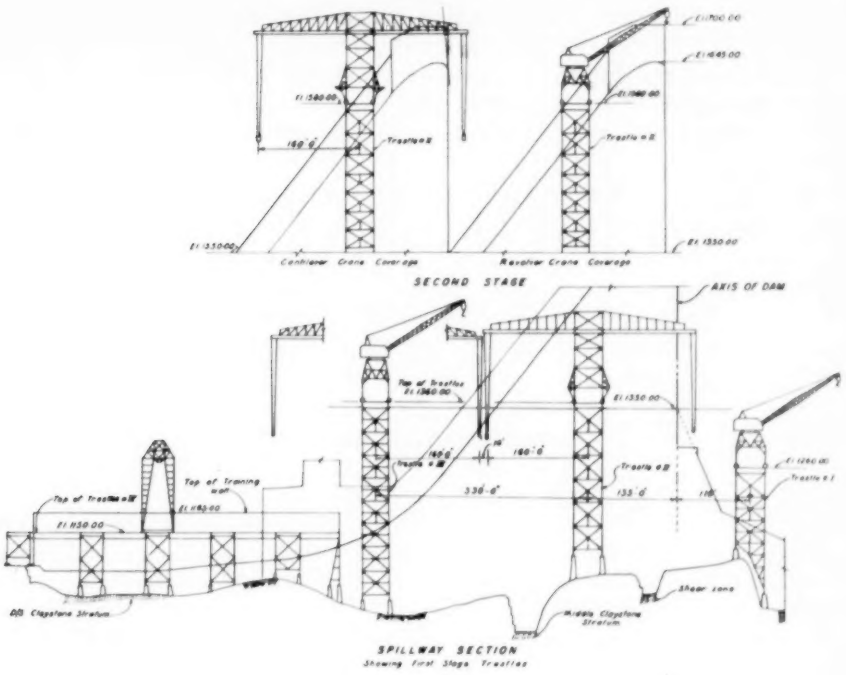

Fig. 8—Trestle system for placement of concrete

DESIGN CONSIDERATIONS AFFECTING CONCRETE MIX

In determining the strength of the mass concrete for larger dams, the usual basis is that the strength should bear a certain ratio to the nominal maximum stress in the dam. Since the stress is a known factor, once the ratio between stress and strength is set, it then becomes a matter for the concrete laboratory to work out the mix. However, besides the strength criteria there are other important considerations such as workability, durability, impermeability, and temperature control, which determine the cement content. Until recently it has been the practice to not go below a 4-bag mix (376 lb per cu yd of concrete). Whereas higher cement content gives greater concrete strength, it also presents problems. The higher the cement content, the greater the heat of hydration and consequent temperature rise, producing greater temperature differential between the concrete mass and the exposure conditions. These conditions are conducive to cracking of concrete and weakening of the structure. It has therefore been felt that the cement content for mass concrete should be the lowest possible, consistent with the desired durability, impermeability, workability, and necessary strength for a reasonable factor of safety.

The maximum computed stresses in Bhakra Dam are expected to be of the order of 800 psi. It has been assumed that the 1-year strength of the concrete shall be four times the nominal maximum stress, giving 1-year concrete strength of 3200 psi. Age strength curves are being established in the laboratories but from data available, it appears that this figure corresponds to a strength of 2500 psi at 28 days.

Studies are continuing to explore the value and practicability of lean mass concrete, starting with 4 bags per cu yd and trying in turn 3, 2½, and 2 bags per cu yd. In all these tests, air-entraining agent (Vinsol resin) was used, sufficient quantity being added to produce 6 ± 0.5 percent of air in concrete made with minus 1½-in. aggregate, which is equivalent to 3.6 ± 0.3 percent in the full mix. Air content was determined by air pressure meter after the mix was wet screened through 1½-in. screen.

CONCRETE INVESTIGATIONS

Laboratory investigations on the proportioning of concrete mixes and other properties of the concrete were made by the U.S. Bureau of Reclamation in 1953-54 and then by the concrete laboratories at Nangal. The tests by USBR were made with Neilla Terrace aggregates as, until the time of making these investigations, it was presumed that about 80 percent of the aggregate would be obtained from that deposit. It was only later that Fatehwal aggregate came into the picture and by that time the concrete laboratories at Nangal had been fully equipped. All mix calculations and other tests at Nangal were, therefore, made on coarse aggregate obtained from Fatehwal and Neilla Terrace combined in the following proportions:

No. 4 to ¾ in.	100 percent Fatehwal
¾ to 1½ in.	24 percent Neilla; 76 percent Fatehwal
1½ to 3 in.	25 percent Neilla; 75 percent Fatehwal
3 to 7 in.	29 percent Neilla; 71 percent Fatehwal

The fine aggregate was entirely from Fatehwal. The laboratory tests proved that there was practically no difference in the aggregates obtained from the two sources as regards their concrete making qualities. The conclusions based on tests at the U.S. Bureau of Reclamation and Nangal will, therefore, be discussed together.

At Nangal laboratories, 12 concrete mixes were proportioned with four set cement factors of 2, 2½, 3, and 4 bags per cu yd, and three economically feasible fineness moduli of Fatehwal sand: 2.25, 2.50, and 2.65. Proportions of various ingredients were established after a large number of trial batches. The materials were mixed in a 7-cu ft tilting-drum mixer. The aggregate, cement, and mixing water were precooled to give an initial concrete temperature of 62-65 F.

Whereas the USBR laboratories used aggregate of 8 in. maximum size, a

maximum size of 7 in. was used at Nangal. Complete gradings used by the two laboratories are given in Table 3.

Due to difference in the grading of the aggregates, cement content required for a given strength or alternatively the strength of concrete for a fixed amount of cement, water-cement ratio for the same slump, and sand-aggregate ratio are bound to vary, but these tests proved conclusively that there is no difference firstly, between the coarse aggregates from Neilla and a mixture of coarse aggregate from Fatehwal and Neilla, and secondly between the fine aggregates obtained from the two sources.

Small changes in the cement content and aggregate gradings will not greatly affect the thermal properties, drying shrinkage, permeability, and thermal expansion of concrete.

TABLE 3—AGGREGATE GRADING FOR CONCRETE TESTS

Screen size	USBR	Nangal
Coarse aggregate		
No. 4 to ¾ in.	9	22
¾ to ¾ in.	13	
¾ to 1 ½ in.	23	18
1 ½ to 3 in.	25	25
3 to 6 in.	27	35
6 to 8 in.	3	
Fine aggregate		
No. 4	—	1.5
No. 8	12.0	15.0
No. 16	11.0	11.0
No. 30	27.0	13.0
No. 50	20.0	31.5
No. 100	25.0	20.0
Pan+200	5.0	8.0
Fineness modulus	2.5	2.5

Summary of test results

The test results indicate the following conclusions:

1. The cement, IS 269-51 (corresponding to ASTM Type I), and the aggregate from Neilla and Fatehwal will produce excellent concrete.

2. A mix containing 3 sacks of cement per cu yd will produce concrete of adequate strength, over 3200 psi at 1 year (8 x 16-in. test cylinders) for the interior mass concrete and will have the following characteristics:

(*a*) *Thermal properties*—The concrete has high thermal conductivity and diffusivity resulting from a high percentage of quartz in the aggregate. At 70 F, the diffusivity is 0.070 sq ft per hr, whereas an average for USBR projects is 0.042 sq ft per hr. The specific heat is 0.22 Btu per lb per deg which is about average for mass concrete. The cement has a heat of hydration of 97 calories per g and produced adiabatic temperature rise in the concrete of 52.8 F. The coefficient of thermal expansion, 0.0000064 in. per in. per deg F is slightly higher than average.

(*b*) *Elastic properties*—The modulus of elasticity at 1-year age is approximately 4,000,000 psi for 18 x 36-in. cylinders and 5,000,000 psi for 6 x 12-in. cylinders.

(*c*) *Drying shrinkage*—The unit drying shrinkage at 1-year age, 0.000390 in. per in., is about the same as the average for several large United States dams.

(*d*) *Permeability*—The permeability coefficient, based on tests of 18 x 18-in. cylinders of the full mix, is 0.055 cu ft per sq ft per year for unit hydraulic gradient. This is for 8-in. mass concrete. The permeability rate is higher than for

TABLE 4—COMPARISON OF BHAKRA DAM CONCRETE WITH UNITED STATES DAMS*

Structure	Bhakra Dam	Hoover Dam	Grand Coulee Dam	Shasta Dam	Hungry Horse Dam	Canyon Ferry Dam
Cement						
Type	I	IV	IV	IV	II	II
Content, sacks per cu yd	3	4	4	4	3†	3‡
Compressive strength, 6 x 12-in. standard cylinders, fog cured, psi						
28-day	3020	3440	4190	3200	2680	3140 (2610)§
90-day	3670	—	6660	5320	4100	4480 (3900)
1-year	4000	—	—	5870	5270	5150 (4630)
Adiabatic temperature rise, deg F	53	50	51	59	44	48
Thermal properties						
Specific heat, Btu per lb per deg F	0.22	0.22	0.22	0.22	0.22	0.22
Conductivity, Btu per ft per hr per deg F	2.3	1.7	1.1	1.3	1.7	1.6
Diffusivity, sq ft per hr	0.070	0.050	0.031	0.037	0.051	0.049
Length change characteristics, 4 x 4 x 30-in. bars						
Drying shrinkage, unit length changes in millionths	390	270	420	—	520	390
Autogenous shrinkage, unit length change in millionths	29	—	—	—	50	45
Coefficient of thermal expansion, unit length change, 4 x 4 x 30-in. bars, millionths per deg F	6.4	5.1	4.4	—	6.3	6.0
Modulus of elasticity at 1 year, psi $\times 10^{-6}$						
6 x 12-in. cylinders	4.9	—	—	—	5.5	5.2
18 x 36-in. cylinders	3.7	—	—	—	5.0	5.6
4 x 4 x 30-in. bars	4.2	5.8	5.9	—	4.6	5.3
Permeability, cu ft per sq ft per year unit hydraulic gradient $\times 10^{4}$						
18 x 18-in. cylinders, 180 days	55.3					1.9
6 x 6-in. cylinders, 180 days	1.7				3.6	2.8
Flexural strength, 4 x 4 x 30-in. bars, psi	500	650	740	—	550	555

*USBR Concrete Laboratory Report No. C-737.

†68 percent portland cement, 32 percent fly ash.

‡70 percent portland cement, 30 percent fly ash.

§The compressive strengths in parentheses for Canyon Ferry Dam are for concrete containing 2½ sacks of cementing material per cu yd (75 percent portland cement and 25 percent fly ash).

Canyon Ferry and Hungry Horse dams but is considered satisfactory for this size of aggregate.

(*e*) *Durability*—Freezing and thawing durability tests were made on 3 x 6-in. cylinders, with material larger than ¾ in. wet-screened out of the mix. Over 1100 cycles of freezing and thawing for a 3-bag mix (90 days fog cured) were required to produce the 25 percent weight loss, which is considered to be excellent.

(*f*) *Workability*—The workability of concrete was good. There was little bleeding or segregation. The concrete is expected to be easy to handle and place.

Test values of 3-sack per cu yd mass concrete for Bhakra Dam are compared with average values for several large U.S. Bureau of Reclamation dams in Table 4.

AGGREGATE TESTING

Both the Fatehwal and Neilla aggregate were subjected to exhaustive tests including petrographic, abrasion, sodium sulfate, freezing and thawing, dura-

bility, and alkali-aggregate reactivity. Complete tests on Neilla aggregates were performed by the U. S. Bureau of Reclamation and results presented in *Concrete Laboratories Report* No. C-626. Tests on Fatehwal aggregates were performed by the Punjab government's modern concrete laboratories at Nangal set up for the project. The petrographic examination of Fatehwal aggregates was done by the Geological Survey of India. Lithologically and in their physical character, the aggregate materials at Neilla and Fatehwal are identical. The main difference is in their physical gradings; while the Neilla aggregate abounds in higher fractions, that at Fatehwal has excess in finer grades. Petrographic examination of these aggregates shows that the gravel particles are made mostly of hard, nonporous material, generally round. Individual pieces are generally smooth, physically sound, and durable. A small proportion contains a coating of calcareous matter which is easily removed by washing.

The gravel is composed predominantly of quartzite, quartzitic sandstone, greyish sandstone, and limestone, but a minor portion is formed of chalcedony, chert, flint and geneissose phyllite, and tourmaline granite.

Quartzite forms more than 85 percent of the whole mass. The sandstones vary widely from a fairly hard and compact greyish sandstone to a highly porous fragile limonitic calcareous type.

The sand is mainly formed of quartzite of light to dark grey color, smooth surfaced, and sub-rounded shape.

The percentage of deleterious and physically unsound constituents is within permissible limits both for gravel and sand.

Los Angeles abrasion test

The Los Angeles abrasion percent loss was 2.2 at 100 revolutions and 14.3 at 500 revolutions. The gravel was given an "A" grading (as per Designation 21, USBR *Concrete Manual*). These results show that the aggregate is extremely hard and resistant to abrasion. Also the breakdown of the aggregate during stockpiling and handling may be expected to be less than usual.

Sodium sulfate soundness

The results of sodium sulfate soundness test on the sand and gravel for Neilla are given in Table 5. The sand and gravels show extremely low losses in this test.

Freezing and thawing durability

The relative durability of concrete subjected to freezing and thawing was measured in the laboratory by an accelerated freezing and thawing test. Although freezing and thawing conditions may not be encountered at the dam site, this test gives a comparison of the relative quality of concrete made with the project aggregate and concrete made with aggregate that is known to be of good quality.

Test cylinders, 3 x 6 in., cast from mixes based on 0.51 water-cement ratio and approximately 3-in. slump, were subjected to alternate cycles of freezing

TABLE 5—SOUNDNESS OF AGGREGATE: SODIUM SULFATE SOUNDNESS TESTS

Aggregate	Screen size	Typical grading, percent	Weighted percent loss, 5 cycles
Sand	No. 8	20	0.5
	No. 16	20	0.5
	No. 30	30	0.7
	No. 50	30	0.5
	Total	100	2.2
Coarse	¾ in.	50	0.4
	¾ in.	30	0.1
	No. 4	20	0.2
	Total	100	0.7

and thawing until 25 percent of the original weight was lost, which is considered failure. One mix was made without entrained air and the other contained approximately 5.5 percent entrained air.

Concrete made without entrained air withstood 250 cycles of freezing and thawing before failure occurred. The test on the air-entrained concrete was discontinued after 1180 cycles with only 14 percent weight loss. The 28-day compressive strengths of the plain and air-entrained concretes were 5720 and 4080 psi, respectively.

Chemical test of alkali reactivity

Representative portions of the samples of gravel and sand were subjected to the chemical test for alkali reactivity. The results indicate that both gravel and sand reactivity would be innocuous.

Conclusions

The aggregates have excellent physical properties and are suitable for use in concrete. Results of petrographic examination and chemical test for alkali reactivity indicate these aggregates will not cause deleterious expansion in concrete because of chemical reaction with alkalies in cement. A summary of quality of coarse aggregate (Neilla) is given in Table 6.

The grading of the coarse aggregates from the two deposits is such that practically no crushing of the aggregates will be necessary.

TESTS OF POZZOLANIC ADMIXTURES

Use of pozzolanic admixtures to replace a part of the cement in concrete has become increasingly popular in recent times. The Bhakra aggregate does not require the use of a pozzolanic admixture to inhibit alkali-aggregate reactivity; nevertheless, other substantial benefits result from use of a pozzolanic admixture in mass concrete. Replacement of 15 to 20 percent cement by a suitable material helps to improve impermeability, workability, and durability of concrete and holds down temperature rise due to heat of hydration. There are savings in the quantity of cement required and hence in the cost of concrete. This economic factor, is particularly important for India where a large number of projects are in progress of development, and cement is in short supply.

Shale and "surkhi"

For Bhakra Dam, two pozzolanic materials were considered: *surkhi* (pulver-

TABLE 6—SUMMARY OF QUALITY OF COARSE AGGREGATE (NEILLA)

Degree of quality		Percent by weight	
		$1\frac{1}{2}$–$\frac{3}{4}$ in.	$\frac{3}{4}$–$\frac{3}{8}$ in.
Physical quality	Good	25.1	24.2
	Satisfactory	74.3	71.9
	Fair	0.6	2.8
	Poor	—	1.1
Chemical quality	Innocuous	96.4	98.9
	Deleterious*	3.6	1.1

*Deleterious with high-alkali cement.

ized burnt brick) and shale.

A shale deposit, sufficient to meet the dam requirements, is about ½ mile upstream from the dam site. The deposit is, however, not uniform. It is interspersed with alternating siltstone strata constituting about 10-15 percent of the shale, and it may not be possible to eliminate entirely the siltstone during excavation. Laboratory tests do not indicate any adverse effects due to presence of this siltstone on calcining, grindability, or pozzolanic properties of shale.

Laboratory tests

Large samples of shale were collected and calcined in the laboratory at various temperatures. The calcined shale was then pulverized in the laboratory and used for testing.

The following tests were performed: (1) fineness, (2) chemical analysis, (3) optimum temperature of calcination, (4) mix proportioning studies and optimum replacement of cement, (5) modulus of elasticity, (6) adiabatic temperature rise studies, and (7) drying shrinkage, and long range volume change studies.

The fineness and chemical analysis of the two materials after pulverizing is given in Table 7. The corresponding specifications for calcined shale used at Davis Dam in the United States are given for comparison.

The results of these tests show that:

1. It is easy to grind calcined shale to a fineness even greater than that of cement. At Davis Dam, where calcined shale was used as pozzolan, a minimum specific surface of 8000 sq cm per g was specified although an average of 12,710 sq cm per g was obtained.

2. Due to high silica and low calcium oxide content, the two materials seem to be promising for use as pozzolan in Bhakra concrete.

3. Calcined shale improved workability of concrete without increase in water-cement ratio. In the case of *surkhi*, however, more water was required to get the same slump than without the additive.

4. Incorporation of shale up to 20 percent of cement reduced bleeding of concrete as compared with plain mix, whereas the bleeding increased in the case of *surkhi*.

5. Although there is a slight decrease in strength at early ages of concrete

TABLE 7—POZZOLAN DATA

	Calcined shale	Surkhi	Davis Dam pozzolan
Fineness, specific surface, sq cm per g	4300	1800	8000 (minimum)
SiO_2, percent	69.4	74.4	60 (minimum)
Fe_2O_3, percent	5.4	9.6	2 (minimum)
Al_2O_3, percent	19.2	7.88	15 (minimum)
CaO, percent	2.3	2.32	10 (minimum)
MgO, percent	1.08	3.92	4 (maximum)
Loss on ignition, percent	1.30	0.24	7 (maximum)
Specific gravity	2.6–2.7	—	2.3 (minimum)

made with either of the pozzolans there is practically no loss in ultimate strength. Regain of strength in the case of *surkhi* is, however, slower than in the case of calcined shale.

6. The adiabatic rise in temperature of concrete with calcined shale is appreciably less than that with neat cement.

7. The volume change of concrete due to drying shrinkage is generally less with the incorporation of calcined shale and *surkhi* as compared with neat cement at all ages.

Production of calcined shale

The job on hand now is to find a way to manufacture 150-200 tons daily of calcined shale of the desired quality. The main problem is effecting an adequate and uniform degree of calcination so that proper pozzolanic activity will be induced and so that the physical properties such as grindability and water requirements will be sufficiently uniform. Suitable rotary kilns are being devised in the Nangal workshop to calcine shale at 1400 F for 2 hr. Results from small scale models were satisfactory and larger kilns are now being developed.

It was considered likely that pozzolan might not be available for use immediately when concreting operations started. Question naturally arises if having two types of concrete, one with pozzolan and the other without it, will produce a satisfactory job. Tests show that there is no significant difference in moduli of elasticity with the 20 percent replacement of cement by calcined shale. Therefore, it is believed that the mass can be assumed to behave as a homogeneous isotropic material for all practical purposes.

Differential expansion and shrinkage

Two other factors needing serious consideration before finally accepting the use of calcined shale in Bhakra Dam after it has been partially built are: (a) difference in thermal coefficients of expansion of the two concretes (This is capable of accurate determination in the laboratory. However, no difference of consequence is expected in the thermal coefficients of the two concretes.); and (b) difference in autogenous shrinkage, which may cause differential opening of the joints, thereby setting up critical changes in stresses and shear friction factors of safety.

Autogenous shrinkage takes place to some degree in many portland pozzolan

cement concretes. This is shrinkage which occurs from spontaneous causes over a long period even though the moisture content and temperature do not change.

The shrinkage observations of mortar bars kept at constant temperature and humidity conditions is continuing and further data are being collected. Observations made up to periods of 6 months have not indicated any cause for concern on this account.

If autogenous shrinkage were anticipated, it would be possible to counteract the effect to some extent by subcooling the concrete a definite amount below the final stable temperature, before grouting of the contraction joints. Complete counteraction by this means could not be expected because the time and space patterns of temperature rise would not be the same as the pattern of autogenous shrinkage.

ACKNOWLEDGMENTS

For the construction of Bhakra Dam an effort has been made to draw upon the experience of the U. S. Bureau of Reclamation, Tennessee Valley Authority, U. S. Army Corps of Engineers, and the private engineering industry, from which sources some of the specialist personnel have been recruited for the project. A board of consultants has been set up to review the more important technical issues at the site and to advise the project authorities.

The authors have been associated with the Bhakra Nangal project for a number of years, having worked in the Central Designs Organization and the Research Institute connected with the project. Much of the information for this paper comes from individuals in direct charge of construction, design, and research for this project.

Opinions expressed are those of the authors and do not constitute any official appraisal.

For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

Title No. 53-11

An Expression for Creep and Its Application to Prestressed Concrete*

By CEVDET Z. ERZEN†

SYNOPSIS

A simple expression for total strain (elastic plus creep) is given. The equation for the total strain is then used in determining the loss of stress in the prestressed beams due to creep of concrete. Resulting analysis indicates that the losses in the prestressed beams may be calculated if the variation of modulus of elasticity of concrete with time and the creep equation of concrete are known.

INTRODUCTION

Recent developments in the theory and practice of prestressed concrete have resulted in a more thorough study of the problem of creep. One of the most significant factors influencing the behavior of prestressed beams is the creep of concrete. Since the behavior of concrete under sustained loads is manifested in the change of stress, it is necessary to study the problem by considering time as one of the independent variables. The difficulty in such an analysis lies in the complexity of the mathematical analysis and the resulting integral equations which become still more difficult to solve due to the complicated nature of the existing creep equations. A further complication arises because of the change of modulus of elasticity of concrete with time.

In analyses so far advanced, all these different factors were not taken into account. It seems only reasonable to start the study of the creep effect of concrete on prestressed beams by developing a simple expression for creep so as to simplify the final analysis to some extent.

AN EXPRESSION FOR TOTAL STRAIN

The quantities which are considered to affect the total strain, ϵ_t, may be taken as

σ = unit stress
E_K = modulus of elasticity of concrete at time of loading
K = time of loading measured from the day of casting of concrete
t = time at which total strain is measured (origin of time taken at time of casting of concrete)

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†Designer, Ammann and Whitney, New York, N. Y. Formerly, Assistant Professor of Civil Engineering, Cornell University, Ithaca, N.Y.

The total strain is the sum of the instantaneous elastic strain ϵ_K and the creep strain ϵ_{cr} taking place during the time $\Delta t = t - K$. In the dimensionless form, the total strain may be represented by

$$\epsilon_t = \frac{\sigma}{E_K} + f\left(\frac{\sigma}{E_K}, \frac{K}{t}\right)$$

where ϵ_{cr} is considered to be a function of ϵ_K and K/t. Assuming that the creep strain varies linearly with the initial unit stress, the above expression may be written as

$$\epsilon_t = \epsilon_K\left[1 + \phi\left(\frac{K}{t}\right)\right]$$

or

$$\epsilon_t = \epsilon_K F\left(\frac{K}{t}\right)$$

If σ is taken as unity, then the above expressions will be due to a unit stress. Henceforth, ϵ_t and ϵ_K will denote strains due to $\sigma = 1$ psi.

The equation for the total strain may be written in the dimensionless form

$$\frac{\epsilon_t}{\epsilon_K} = e^{\alpha\left[1-\left(\frac{K}{t}\right)^{\beta}\right]} \qquad \text{(1)}$$

in which α and β are constants and are determined from test results. This equation contains both the elastic and plastic strains since substitution of $t = K$ reduces the total strain to the instantaneous or elastic strain ϵ_K. It may also be observed that the total strain at $t = \infty$ is constant and is given by

$$\epsilon_\infty = e^{\alpha}\epsilon_K$$

This indicates that the ratio of the total strain (at $t = \infty$) to the instantaneous strain is constant, which fact was first noted by Freyssinet and then confirmed by the tests conducted by Le Camus.[1]

Eq. (1) for the Shasta Dam[2] concrete takes the form

$$\frac{\epsilon_t}{\epsilon_K} = e^{0.75\left[1-\left(\frac{K}{t}\right)^{0.75}\right]}$$

which is plotted in Fig. 1 together with the test results for different time of loading.

It is assumed that $\epsilon_K = 1/E_K$ is known for different ages of concrete. This may be determined by tests. Therefore, in order that total strain be calculated from Eq. (1), values of instantaneous strain or modulus of elasticity must be known at the time of loading. In the absence of such data, since the concrete strength is based on the 28-day tests, the modulus of elasticity for ordinary portland cement concrete may be found approximately from the writer's dimensionless formula

$$\frac{E_t}{E_{28}} = \frac{4}{3 + \frac{28}{t}} \qquad \text{(2)}$$

For high-early-strength concrete this formula is modified to take into account the rapid increase of the modulus of elasticity. This may be written as

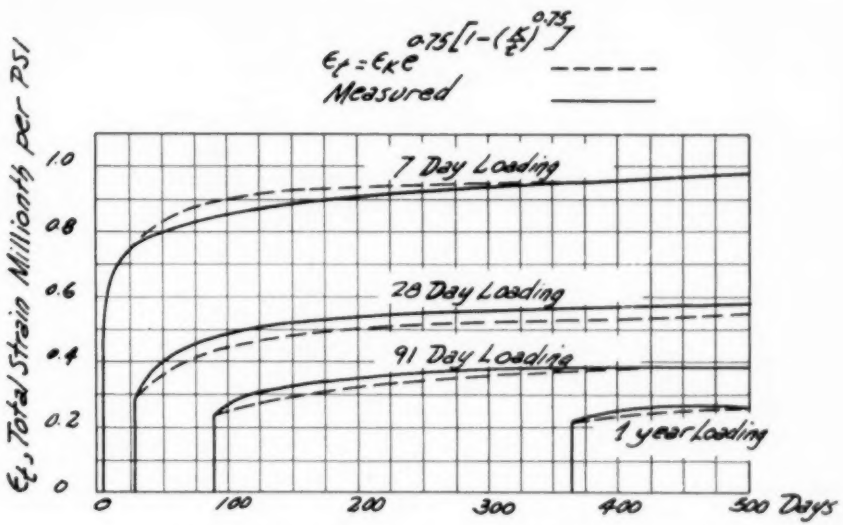


Fig. 1—Elastic plus creep strain—Shasta Dam concrete

$$\frac{E_t}{E_{28}} = \frac{12}{11 + \frac{28}{t}} \qquad (3)$$

In Fig. 2, Eq. (2) and (3) and the measured values of Shasta Dam[2] concrete are plotted for comparison.

LOSS OF STRESS IN PRESTRESSED BEAM DUE TO CREEP

The loss of tension in the wires of a prestressed beam occurs in two stages. In the first stage the loss is due to the instantaneous strain upon transfer of the stress from the wires to the concrete. This loss occurs because of the elasticity of concrete and steel and is denoted by P_{tr} and is known as loss on transfer. However, once the load is transferred to the concrete, creep action takes place and then the initial stress in the section is modified.

The unit strain in the wires due to elastic loss upon transfer is

$$\frac{P_{tr}}{A_s E_s}$$

where A_s and E_s are the area and modulus of elasticity of the steel wires, respectively. If the initial stress in the wires is P before transfer of stress, and K is the age of concrete at time of loading, then equating the unit strain in concrete at the position of the center of gravity of wires, there is found

$$\frac{P_{tr}}{A_s E_s} = \frac{P - P_{tr}}{A_s E_K}\left(1 + \frac{c^2}{r^2}\right) - \frac{M_D c}{E_K I}$$

Solving for P_{tr} from above

$$P_{ir} = \frac{\left(P - \frac{M_D c}{r^2 + c^2}\right)\left(1 + \frac{c^2}{r^2}\right) n_K p}{1 + \left(1 + \frac{c^2}{r^2}\right) n_K p} \qquad (4)$$

where

M_D = dead load moment
c = distance from centroidal axis of section to center of gravity of wires
r = radius of gyration of section
$p = A_s/A_c$
$n_K = E_s/E_K$

In the second stage, creep losses become apparent due to the change of strain

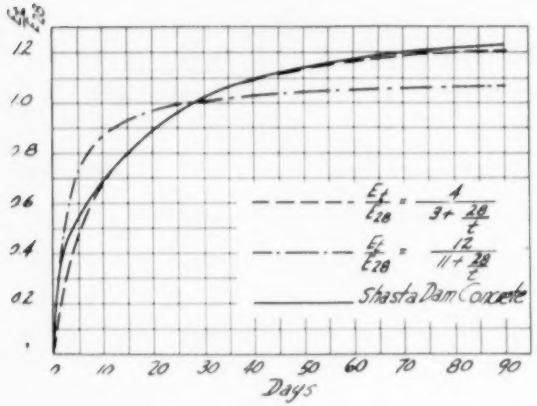


Fig. 2—Variation of modulus of elasticity with time

with time. The initial strain in concrete right after the application of the load produces a total strain at time t given by the expression

$$\epsilon_1 = \frac{1 + e^2/r^2}{A_c E_K}\left(P - P_{ir} - \frac{M_D c}{r^2 + c^2}\right) e^{n\left[1 - \left(\frac{K}{t}\right)^{\beta}\right]} \qquad (a)$$

Because of the change of strain there results a continuous loss of stress denoted by $\bar{P}(t)$, as shown in Fig. 3a, such that the total loss at any time is given by

$$P_L = P_{ir} + \bar{P}(t)$$

Thus, as the strain due to initial stress takes place, there results a different strain which corresponds to $\bar{P}(t)$ and is given by

$$\epsilon_2 = \int_K^t e^{n\left[1 - \left(\frac{\xi}{t}\right)^{\beta}\right]} \frac{d\bar{\epsilon}(\xi)}{d\xi}\, d\xi \qquad (b)$$

in which

$$\frac{d\bar{\epsilon}(\xi)}{d\xi}\, d\xi$$

is the increment of strain due to $\bar{P}(\xi)$ during $d\,\xi$ whose contribution to the total strain at time t is given by Eq. (b).

Therefore the net unit total strain in the concrete, which is also the unit strain in the steel, at time t is the difference in the strains ϵ_1 and ϵ_2. Hence

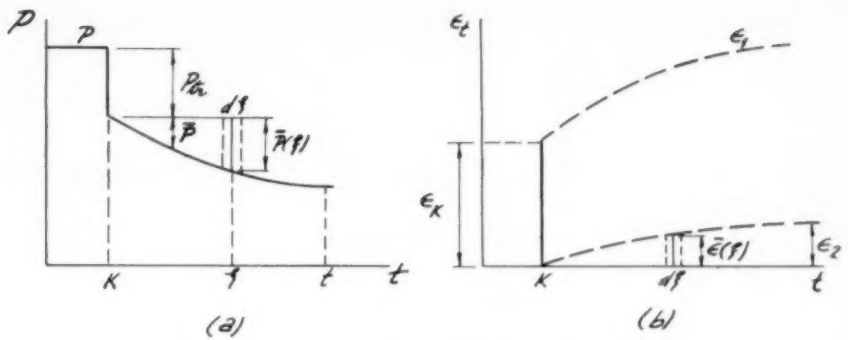


Fig. 3—Variation of stress and strain with time

$$\frac{P_L}{A_c E_c} = \frac{1+\frac{c^2}{r^2}}{A_c E_K}\left(P - P_{tr} - \frac{M_D c}{r^2+c^2}\right) e^{\alpha\left[1-\left(\frac{K}{t}\right)^{\beta}\right]} - \int_K^t e^{\alpha\left[1-\left(\frac{\xi}{t}\right)^{\beta}\right]} \frac{d\bar{\epsilon}(\xi)}{d\xi} d\xi$$

The second term on the right hand side is integrated by parts as

$$-\left[e^{\alpha\left[1-\left(\frac{\xi}{t}\right)^{\beta}\right]}\bar{\epsilon}(\xi)\right]_K^t - \frac{\alpha\beta}{t^{\beta}}\int_K^t e^{\alpha\left[1-\left(\frac{\xi}{t}\right)^{\beta}\right]}\xi^{\beta-1}\bar{\epsilon}(\xi)d\xi$$

since

$$\bar{\epsilon}(K) = 0$$

and

$$\bar{\epsilon}(\xi) = \frac{\bar{\sigma}(\xi)}{E\xi} = \frac{\bar{P}(\xi)}{E\xi A_c}\left(1+\frac{c^2}{r^2}\right)$$

There results

$$\frac{P_L}{A_c E_c} = \frac{1+\frac{c^2}{r^2}}{A_c E_K}\left(P - P_{tr} - \frac{M_D c}{r^2+c^2}\right) e^{\alpha\left[1-\left(\frac{K}{t}\right)^{\beta}\right]} - \frac{\bar{P}(t)}{A_c E_t}\left(1+\frac{c^2}{r^2}\right)$$
$$- \frac{\alpha\beta}{t^{\beta}} \frac{1+\frac{c^2}{r^2}}{A_c}\int_K^t e^{\alpha\left[1-\left(\frac{\xi}{t}\right)^{\beta}\right]}\xi^{\beta-1}\frac{\bar{P}(\xi)}{E\xi}d\xi \qquad (5)$$

which is an integral equation containing the unknown function $\bar{P}(t)$. The solution of this equation may be obtained by the method of successive substitutions which entails successive integrations. The solution then is found in terms of an infinite series. However, the integration encountered in this solution is much too difficult to perform and therefore this method is abandoned in favor of a numerical solution which reveals the nature of the function for P_L.

In carrying out the numerical solution the last term of Eq. (5) is transformed from an integral to a series as,

$$-\frac{\alpha\beta}{t^{\beta}}\frac{1+\frac{c^2}{r^2}}{A_c}\sum e^{\alpha\left[1-\left(\frac{\xi}{t}\right)^{\beta}\right]}\xi^{\beta-1}\frac{\bar{P}(\xi)}{E\xi}\Delta\xi$$

in which $p(\xi)$ is the loss of stress as shown in Fig. 3a. The method of solution is illustrated in the following example.

Given a beam that possesses the following properties:

$A_c = 203$ sq in.	$I = 18{,}350$ in.^{4}
$A_s = 4.52$ sq in.	$r^2 = 90.5$ sq in.
$p = 0.0223$	$E_c = 29 \times 10^6$ psi
$e = 11$ in.	$P = 192{,}000$ lb

If the beam is considered lying on its side, $M_D = 0$. Taking the Shasta Dam concrete loaded on the seventh day

$$E_7 = 2.11 \times 10^6 \text{ psi}, \; n_7 = 13.7$$

Therefore from Eq. (4)

$$P_u = \frac{0.0223 \times 13.7\left(1 + \frac{121}{90.5}\right)P}{1 + 0.0223 \times 13.7\left(1 + \frac{121}{90.5}\right)} = 0.417P = 80{,}000 \text{ lb}$$

To find the loss on the succeeding days Eq. (5) is used. Taking $\beta = 1$, there is found

$$\frac{P_u + \overline{P}(t)}{p\left(1 + \frac{e^2}{r^2}\right)E_c} = \frac{P - P_u}{E_K} e^{-\left[1 - \frac{K}{t}\right]} - \frac{\overline{P}(t)}{E_t} - \frac{\alpha}{t}\sum e^{-\left(1 - \frac{\xi}{t}\right)} \frac{\overline{P}(\xi)}{E\xi} \Delta\xi$$

Substituting the numerical values

$$0.660(P_u + \overline{P}(t))10^{-6} = 52900 \times 10^{-6} e^{-\left(1 - \frac{K}{t}\right)} - \frac{\overline{P}(t)}{E_t} - \frac{0.74}{t}\sum e^{-\left(1 - \frac{\xi}{t}\right)} \frac{\overline{P}(\xi)}{E\xi} \Delta\xi$$

in which $\overline{P}(\xi)\ \Delta\xi$ is the differential area as shown in Fig. 3a. To find the additional loss on the eighth day, in the above equation t and ξ are taken as 8 and 7.5, respectively, and

$$\overline{P}(7.5)\Delta\xi = \tfrac{1}{2}\overline{P}(8) \times 1$$

Since

$$\frac{1}{E_8} = 0.460 \times 10^{-6}, \; \frac{1}{E_{7.5}} = 0.466 \times 10^{-6}$$

the equation becomes, with $\alpha = 0.74$

$$0.660(\overline{P}_8 + 80000) = 58000 - 0.460\overline{P}_8 - 0.0925 \times 1.0477 \times \frac{1}{2} \times \overline{P}_8 \times 0.466$$

Solving for $\overline{P}_8$ there is found

$$\overline{P}_8 = 4540 \text{ lb}$$

Similarly for $t = 9$

$$\frac{1}{E_9} = 0.448 \times 10^{-6}, \; \frac{1}{E_{8.5}} = 0.454 \times 10^{-6}$$

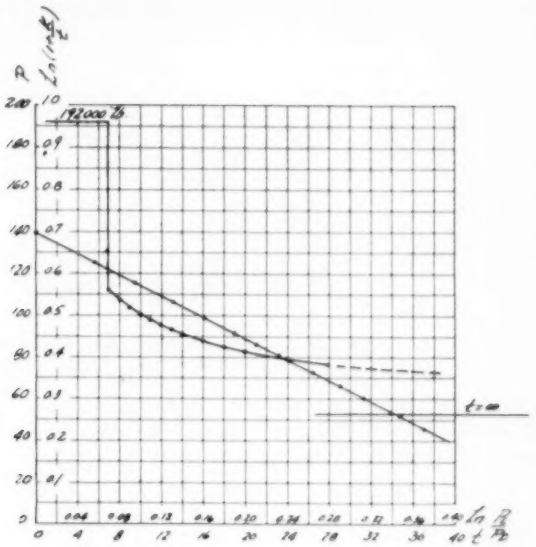
$$0.660(\overline{P}_9 + 80{,}000) = 52{,}900 \times 1.178 - 0.448\overline{P}_9 - 0.0823\left(\frac{1}{2}\,4540 \times 1.131 \times 0.466 + \frac{4540 + \overline{P}_9}{2} \times 1.0856 \times 0.454\right)$$

where ½ (4540 × 1) and $[(4540 + \overline{P}_9)/2] \times 1$ are the two consecutive differential areas.

Solving

$$\overline{P}_9 = 8430 \text{ lb}$$

Fig. 4—Variation of stress with time



Similar equations are set up to find the loss $\overline{P}(t)$ on the succeeding days. These computations are tabulated in Table 1 and are plotted in Fig. 4.

Also tabulated in Table 1 are $L_n\ (1 + k/t)$ versus $L_n\ (P_L/P_{tr})$. It is seen in Fig. 3 that these values fall on a straight line. If the equation of this line is expressed algebraically the following expression is found

$$P_L = \frac{1.74}{(1 + 7/t)^{0.8}} P_{tr}$$

Thus, from the above formula the loss in steel stress of the beam given in the example may be found at any time.

The linear relationship between $L_n\ (1 + K/t)$ and $L_n\ (P_L/P_{tr})$ makes it possible to determine a general expression for the function P_L. If in Eq. (5) t is taken at $K + 1$, there is found

$$\frac{A}{E_c}(P_{tr} + \overline{P}_{K+1}) = \frac{B}{E_K} - \frac{\overline{P}_{K+1}}{E_{K+1}} - C\frac{\overline{P}_{K+1}}{E_{K+\frac{1}{2}}}$$

where

$$A = \frac{1}{p(1 + e^2/r^2)} \qquad (6)$$

$$B = \left(P - P_{tr} - \frac{M_D e}{r^2 + e^2}\right) e^{-\alpha\left[1-\left(\frac{K}{K+1}\right)^{\beta}\right]} \qquad (7)$$

$$C = \frac{\alpha\beta}{2(K+1)^{\beta}} e^{-\alpha\left[1-\left(\frac{K}{K+1}\right)^{\beta}\right]} \left(K + \frac{1}{2}\right)^{\beta-1} \qquad (8)$$

Solving for $\overline{P}_{K+1}$

TABLE 1—LOSSES OF PRESTRESS WITH TIME

t	$\bar{P}$ (t)	P_L	$L_n\left(1+\frac{K}{t}\right)$	$L_n\frac{P_L}{P_{tr}}$
7	0	80000	0.693	0.
8	4540	84540	0.628	0.0573
9	8430	88430	0.575	0.0955
10	11420	91420	0.531	0.1310
11	14000	94000	0.493	0.1612
12	16550	96550	0.459	0.1900
13	18800	98800	0.431	0.210
14	20900	100900	0.405	0.232
16	24100	104100	0.364	0.264
18	26900	106900	0.328	0.290
20	29400	109400	0.300	0.314
24	33100	113100	0.256	0.348
28	35800	115800	0.224	0.370

$$\bar{P}_{K+1} = \frac{n_K B - A P_{tr}}{A + n_{(K+1)} \cdot C + n_{K+1}} \quad \ldots\ldots (9)$$

By substitution it may be shown that $\bar{P}_{K+1} \rightarrow 0$ as K approaches zero. Also, the observed linear relationship may be expressed as

$$L_n \frac{P_L}{P_{tr}} = C_1 L_n\left(1+\frac{K}{t}\right) + C_2$$

From the values of P_L corresponding to $t = K$ and $t = K + 1$

$$C_1 = -\frac{L_n \dfrac{P_{tr} + \bar{P}_{K+1}}{P_{tr}}}{L_n \dfrac{2}{1+\dfrac{K}{K+1}}} \quad \ldots\ldots (10)$$

$$C_2 = -C_1 L_n{}^2 \quad \ldots\ldots (11)$$

Solving for P_L from above

$$P_L = e^{C_2}\left(1+\frac{K}{t}\right)^{C_1} P_{tr} \quad \ldots\ldots (12)$$

which is the desired equation.

To illustrate the use of Eq. (6) to (12), the previous example is considered. Substitution of the data in Eq. (6) to (8) yield $A = 19.18$, $B = 123{,}000$, and $C = 0.0484$. Inserting these values in Eq. (10): $\bar{P}_8 = 4580$ lb; and from Eq. (10) and (11), $C_1 = -0.857$ and $C_2 = 0.593$. Therefore, the resulting equation is

$$P_L = \frac{1.81}{(1+7/t)^{0.857}} P_{tr}$$

In the next example, the same data as in the previous example are used, and the equation for the total loss P_L is found for $\alpha = \beta = 0.75$ and $K = 40$. In this case the modulus of elasticity of concrete is determined from the approximate formula

$$E_t = \frac{4}{3+\dfrac{28}{t}} E_{28}$$

where

$$E_{28} = 3.56 \times 10^6 \text{ psi}$$

Taking the dead load moment, $M_D = 510{,}000$ in.-lb, the loss on transfer from Eq. (4) is $P_{tr} = 46{,}700$ lb. From Eq. (6), (7), (8): $A = 19.18$, $B = 120{,}700$, and $C = 0.00690$. Inserting these constants in Eq. (9): $\overline{P}_{41} = 560$ lb and $C_1 = -0.970$, $C_2 = 0.672$. Finally Eq. (12) becomes

$$P_L = \frac{1.959}{(1 + 40/t)^{0.970}} P_{tr}$$

From which the total loss when $t = \infty$ is found to be $P_L = 91{,}500$ lb.

CONCLUSION

It is shown that the losses due to creep in prestressed concrete beams may be evaluated if the variation of the modulus of elasticity of concrete with time and the creep expression are given. The problem necessitates the solution of an integral equation numerically. The general solution of the problems, however, is made possible once the nature of the function representing the loss of steel stress in the beams is revealed after the numerical solution.

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For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

Title No. 53-12

Empirical Time-Strength Relations of Concrete*

By MYRON L. GORAL†

SYNOPSIS

Modern design concepts and intense competition with other structural materials make it necessary for the contractor of concrete structures to safely improve methods of scheduling concreting operations. Anticipation of the age at which concrete will be sufficiently strong for subsequent operations enables the contractor to plan equipment and personnel in advance of needs and to substantially reduce waiting time. Time-strength forecasting is of particular value in the scheduling of form stripping, removal of shores, handling or erecting precast structural elements, and transferring prestressing forces.

TIME-STRENGTH FACTORS

Factors affecting the rate of strength growth of concrete are well known. Some of these are water-cement ratio, temperature, pressure, admixtures, and quality of the component materials. Equally significant, but less frequently considered, are such factors as uniformity of portland cement from the same or different sources, uniformity of physical and chemical characteristics of the aggregates, and efficiency of curing procedures.

Many types of time-strength curves have been published. In a general way, they show the effects of water-cement ratios, curing conditions, and admixtures. However, these published data provide only a rough approximation of what may be expected from a specific job. Since there is a time lag between the moisture content determination of the aggregates and the corresponding correction in batching, it is not always possible in normal ready-mixed concrete operations to maintain consistently accurate water-cement ratios. Usually, there is a temperature variation in job-placed concrete that is not present in laboratory specimens. Controlling the temperature of job-cast concrete is expensive and is attempted only on the relatively few jobs where unusual conditions justify the additional costs. Unpredictable variables which occur in practice are often costly and difficult to control.

These and other considerations make each concrete placement an individual product with its own time-strength properties. Specifications for most jobs stipulate required compressive tests and modulus of rupture determinations with minimum

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strengths resulting from these tests as necessary conditions to removing shores, tilting up wall panels, transferring prestress, and similar operations which follow casting. If the contractor can estimate the age at which these operations will be permitted, he may properly schedule equipment and personnel. On the other hand, if he must wait until the testing laboratory reports that the required strengths have been reached, there is a needless lag in the schedule from the time the notice is received until equipment and labor can be assembled. It is possible to use standard statistical procedures to estimate when such subsequent operations may be performed.

THEORY

The theory which enables us to forecast the time-strength relation of concrete from early test results, is based upon two assumptions:

1. **For a given concrete mix, under given conditions of mixing, placing, and curing, the concrete strength in compression, flexure, and shear is a function of time.**

2. **At zero time (when the concrete is placed), the strength is zero, *i.e.*, the time-strength curve passes through the origin of a coordinate system.**

If two variables are related, the relation becomes apparent when corresponding points are plotted graphically. The equation of a curve may be obtained by transforming the data, in some manner, so that it plots as a straight line. This transformation into linear form is called rectification. Frequently, we see concrete strength data plotted on logarithmic or semi-logarithmic graph paper. However, the second assumption above eliminates the logarithmic method of rectification since zero is not defined on the logarithmic scale.

We must examine the properties of concrete to obtain a clue to the functional relation between the age of concrete and its corresponding strength. It is well established that, in the absence of deteriorating influences, the strength of concrete increases indefinitely. The nature of the components of portland cement concrete suggests that the strength of concrete is unlikely to increase without limit. Indeed, examination of plots of concrete strengths with respect to time suggests that the growth of concrete strength approaches some limit, but never reaches that limit. The limit a curve approaches, but never reaches, is called an asymptote.

If we reason that the time-strength curve has an asymptotic character, we are led to expect that a hyperbolic curve may well approximate the time-strength relation. Of all the available hyperbolic curves, we choose a curve which passes through the origin and which has a horizontal asymptote. Such a curve may be expressed by

$$f = \frac{t}{b + mt} \qquad (1)$$

where f is the compressive strength in psi and t is its age in days. The strength of concrete at 28 days is denoted by the usual f_c'; b and m are empirical constants. Inspection of Eq. (1) reveals that it satisfies the condition established by the second assumption, namely that it passes through the origin, since $f = 0$ when $t = 0$. The asymptotes of the curve of this equation are:

$t = -b/m$
$f = 1/m$

For rectification, we multiply both sides of Eq.(1) by $(b + mt)$ and divide both sides by f, obtaining

$$\frac{t}{f} = b + mt \quad \text{(2)}$$

It will be noted that the right-hand member of Eq.(2) is identical in form to the right-hand member of the standard slope-intercept form of the equation of a straight line. For simplicity in later computations, let $F = t/f$. Then Eq.(2) becomes

$$F = b + mt \quad \text{(3)}$$

Since the data will be subject to the usual errors of experiments and observations, it is necessary to determine the "line of best fit." The most efficient way to do this is to use the method of least squares. The derivation of this method is found in standard works on the subject (see references) and is not given here. It is not necessary to go through the entire derivation to be able to use the method on this problem. The procedure outlined below is in a form which may be used easily by any contractor's or engineer's office.

Suppose any number, say n, test cylinders are tested. Each test will provide two numbers: (1) age, and (2) strength. The n tests will furnish n pairs of figures: (t_1,f_1), (t_2,f_2), (t_3,f_3), . . . (t_n,f_n). From these, we compute F_1, F_2, F_3, . . . F_n, where $F = t/f$.

Experience shows that if the cylinders are properly made, carefully handled, and precisely tested, then for a series of tests from the same batch of concrete, if t is taken as the independent variable and F as the dependent variable, the points will plot as a straight line on rectangular coordinate paper.

If there were no errors in any of the procedures, we could write the equations:

$F_1 = mt_1 + b$
$F_2 = mt_2 + b$
$F_3 = mt_3 + b$
. . .
. . .
. . .
$F_n = mt_n + b$

and all the equations would have the same numerical values of m and b, respectively. Since errors do occur, it is necessary to find the values of m and b for the line of best fit. The line of best fit is defined as that line which is so located that the sum of the squares of the differences between actual and line values of F will be a minimum. The procedure is quite simple if we introduce the standard notation $\Sigma F = F_1 + F_2 + F_3 + \ldots + F_n$. If we add the corresponding terms of the above n equations, we have

$$\Sigma F = m\Sigma t + nb \quad \text{(4)}$$

Now, if we multiply each term of the original equations by the value of t in each equation, and if we add the corresponding terms, we may write

$$\Sigma tF = m\Sigma t^2 + b\Sigma t \quad \text{(5)}$$

Eq. (4) and (5) are a pair of simultaneous equations which enable us to evaluate

the two unknowns m and b. These equations are called the normal equations of the data. The normal equations are easily solved by the method of determinants. The coefficients of the unknowns on the right-hand side of the equations form the determinant Δ used in evaluating both m and b.

$$\Delta = \begin{vmatrix} \Sigma t & n \\ \Sigma t^2 & \Sigma t \end{vmatrix} = (\Sigma t)^2 - n\Sigma t^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

An expression for Δm is obtained from the determinant formed by substituting the left-hand members of the normal equations for the coefficients of m in the above determinant.

$$\Delta m = \begin{vmatrix} \Sigma F & n \\ \Sigma tF & \Sigma t \end{vmatrix} = \Sigma F \Sigma t - n\Sigma tF \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Similarly, we write the determinant for Δb by substituting the left-hand members of the normal equations for the coefficients of b and expanding the determinant as above, we obtain

$$\Delta b = \Sigma t \Sigma tF - \Sigma t^2 \Sigma F \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Eq. (6), (7), and (8) are used to compute the best values of m and b.

The above equations will provide sufficient information from the test data from a given batch of concrete to enable us to forecast the strength at any age in the immediate future, say $t = 28$ days, or conversely to predict the age at which the concrete will reach a particular strength.

PROCEDURE

Application of the above theory is simple in everyday practice. As soon as two or more tests have been made, the tests are recorded in tabular form as suggested in Table 1.

TABLE 1—FORM FOR RECORDING TEST DATA

t	f	$F = t/f$	t^2	tF
t_1	f_1	F_1	t_1^2	t_1F_1
t_2	f_2	F_2	t_2^2	t_2F_2
t_3	f_3	F_3	t_3^2	t_3F_2
.	.	.	.	.
.	.	.	.	.
.	.	.	.	.
t_n	f_n	F_n	t_n^2	t_nF_n

Step 1

Plot F as the ordinate against t as the abscissa on rectangular coordinate graph paper. If the points plot as a good approximation of a straight line, Eq. (1) is the form of the empirical equation which expresses the relation of time and strength of the concrete.

Step 2

Perform the indicated arithmetical operations in Table 1. The sums of all the columns, except column 2, provide the numbers to be substituted in Eq. (6), (7), and (8) to evaluate m and b.

Step 3

Write Eq. (1) using the proper values of m and b. Compute sufficient values of f for corresponding values of t to enable the plotting of a smooth curve on

rectangular coordinate paper. Plot the actual test data on the same graph for a visual check of agreement between the computed and actual values of f.

The above steps can be repeated using more data as they become available for reforecasting, if necessary.

The result of the three steps will provide accurate information within the time period covered by the tests. As the values of t become larger than the largest observed value of t, the computed value of f will contain an increasingly greater probable error. It is the general function of empirical equations to provide a relation between variables within the range of observed values (see Weierstrass theorems on interpolation). The determination of values of f for corresponding values of t within the interval of observed values is called interpolation. The computation of values of f beyond the range of t actually observed (in the future) is called extrapolation.

Normally, extrapolation is not considered reliable. Extrapolated results must always be viewed as containing certain errors. Usual construction scheduling requires forecasting for the first 28 days of the age of the concrete. Fortunately, all the influences of the quality of a given concrete batch from batching through curing, including the use of admixtures, if any, tend to work together so that the time-strength relation of the concrete may be expressed by Eq. (1) as a first approximation. We do not yet know the manner in which certain influences affect the numerical values of m and b. It is sufficient for our purpose to know that all the influences on the concrete work together in forming the characteristics of a time-strength relation expressed approximately by Eq. (1). Experience shows that if the extrapolation is not too great, the error of the computed values is small enough to permit the preparation of forecasts which are sufficiently accurate for the scheduling of construction operations.

When graph paper is not available for step 3, it is possible to estimate the age, t, at which the concrete will reach a specific strength by analytical methods. This is done by solving Eq. (1) for t in terms of f. We do this by subtracting the term mt from both sides of Eq. (2):

$$\frac{t}{f} - mt = b$$

Now, multiplying both sides of the above equation by f and factoring t, we have

$$t(1 - mf) = bf$$

which permits us now to solve for t by dividing both sides of the last equation by the coefficient of t:

$$t = \frac{bf}{1 - mf} \qquad (9)$$

The product mf must be less than unity. If the product mf is equal to or greater than unity, the chosen strength, f, will never be reached. This means that we are too ambitious for the concrete or that there is an error in the value of m arising either from arithmetical errors or faulty testing.

If some individual test occurs whose values t_i, f_i do not satisfy the function $G(F,t) = 0$, then the test must be viewed with suspicion and must be re-examined

in the light of more data. This criterion enables us to readily identify individual tests which are not reliable and which should be discarded.

EXAMPLE

The wall panels of a precast concrete building were required by the specifications to have a compressive strength of 1600 psi at the time of erection. The concrete contained 5.64 sacks per cu yd of Type I portland cement and the water-cement ratio was 6.47 gal per sack. The maximum aggregate size was 1½ in. Temperatures varied daily, after the concrete was placed, from 24 to 68 F. To counteract the anticipated low temperatures, 2 percent (by weight of the cement) $CaCl_2$ was used as an admixture. Test cylinders provided the following initial data:

Age in days at test, t	Compressive strength, f, psi
2	729
3	900
5	1257

Exact influences of the cold weather and the use of $CaCl_2$ were not well-known. Therefore, in the interest of job safety, the erection of the panels was scheduled for the date at which test cylinders reached 1800 psi.

Problem

(a) Estimate the 7-day strength and find the error when compared with the actual strength. (b) Estimate the age at which the concrete will reach a strength of 1800 psi. (c) Estimate the 14-day strength and compare with the actual strength at that age. (d) Estimate the 28-day strength and compare with the actual 28-day strength.

Solution

Putting the data into the form of Table 1, we have

t	f	F	t^2	tF
2	729	0.002743	4	0.005486
3	900	0.003333	9	0.009999
5	1257	0.003978	25	0.019890
10		0.010054	38	0.035375

The graph of (t,F) does not approximate a straight line. At least one of the three tests contains an error. However, at this stage we do not know which test is not valid. We proceed with the analysis giving equal weight to all the tests, but with the mental reservation that this must be investigated later.

Using Eq. (6) $\Delta = 10^2 - (3 \times 38) = -14$

From Eq. (7) $-14\ m = (0.010054 \times 10) - (3 \times 0.035375) = -0.005585$

$m = +0.000399$

Using Eq. (8) $-14\ b = (10 \times 0.036375) - (38 \times 0.010054) = -0.028302$

$b = +0.002022$

(a) The estimated 7-day strength, using Eq.(1) is

$$f_7 = \frac{7}{0.000399 \times 7 + 0.002022} = 1454 \text{ psi}$$

The actual 7-day strength was reported to be 1500 psi, therefore the error of the estimate is

$$\frac{1500 - 1454}{1454} \times 100 = 3.16 \text{ percent}$$

Curve (a) in Fig. 1 is drawn solid through the actual test data and the extrapolated portion is denoted by a broken line. It will be noted that the curve passes above the 3-day test and below the 2-day test. The rectification line also passes between the 2- and 3-day tests. We now compute F for the 7-day test: $F = 7/1500 = 0.004667$. Plotting this value for $t = 7$, we see that it lies on the straight line passing through the 3- and 5-day tests. Therefore, we assume the 2-day test contains the error suspected above. We discard the 2-day test in later computations for this concrete.

(b) To estimate the age at which the concrete will reach a strength of 1800 psi, we discard the 2-day test and add the 7-day test to the table at the beginning of this solution, obtaining

t	f	F	t^2	tF
3	900	0.003333	9	0.009999
5	1257	0.003978	25	0.019890
7	1500	0.004667	49	0.032667
15		0.011978	83	0.062556

$$\begin{aligned} \Delta &= 15^2 - (3 \times 83) = -24 \\ -24m &= 0.011978 \times 15 - (3 \times 0.062556) = -0.007998 \\ m &= +0.000333 \\ -24b &= 15 \times 0.062556 - (83 \times 0.0011978 = -0.055834 \\ b &= +0.002326 \end{aligned}$$

Now, using Eq.(9), $f = 1800$, and the above values of m and b, we write

$$t = \frac{0.002326 \times 1800}{1 - (0.000333 \times 1800)} = 10.45, \text{ say } 11 \text{ days}$$

Curve (b) in Fig. 1 shows the result of dropping the 2-day test and adding the 7-day test. The two curves are close in the interval of interpolation but begin to diverge significantly when t is greater than 7 days.

(c) Estimate the 14-day strength

$$f_{14} = \frac{14}{(0.000333 \times 14) + 0.002326} = 2003 \text{ psi}$$

The actual 14-day strength was reported to be 2024 psi. Therefore the error of the estimate is

$$\frac{2024 - 2003}{2003} \times 100 = 1.05 \text{ percent}$$

(d) Estimate the 28-day strength. Adding the data of the 14-day test to the table in (b) above, we have

t	f	F	t^2	tF
3	900	0.003333	9	0.009999
5	1257	0.003978	25	0.019890
7	1500	0.004667	49	0.032667
14	2024	0.006917	196	0.096838
29		0.018895	279	0.159394

$$\Delta = 29^2 - (4 \times 279) = -275$$
$$-275m = (0.018895 \times 29) - (4 \times 0.159394) = -0.089621$$
$$m = +0.0003259$$
$$-275b = (29 \times 0.0159394) - (279 \times 0.018895) = -0.649279$$
$$b = 0.0023610$$

To compute the 28-day strength and sufficient points to plot a smooth curve, we use a tabular form suggested by Eq. (1). The first column of the table lists the values of t, which we think will enable us to plot a smooth curve. The second column is the denominator, and the third column is the strength, obtained by dividing each figure in the first column by the corresponding figure in the second column.

t	$mt + b$	f
1	0.0026869	372
2	0.0030128	664
3	0.0033387	899
5	0.0039905	1253
8	0.0049862	1610
12	0.0062718	1913
16	0.0075754	2112
20	0.0088790	2253
24	0.0101826	2357
28	0.0114862	2438

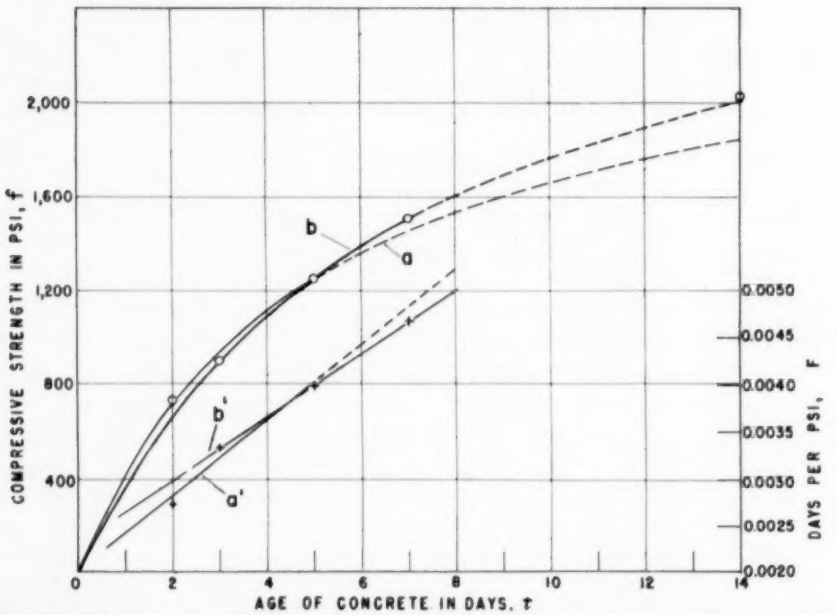

Fig. 1—Effect of rectifying strength-age curve by eliminating questionable data

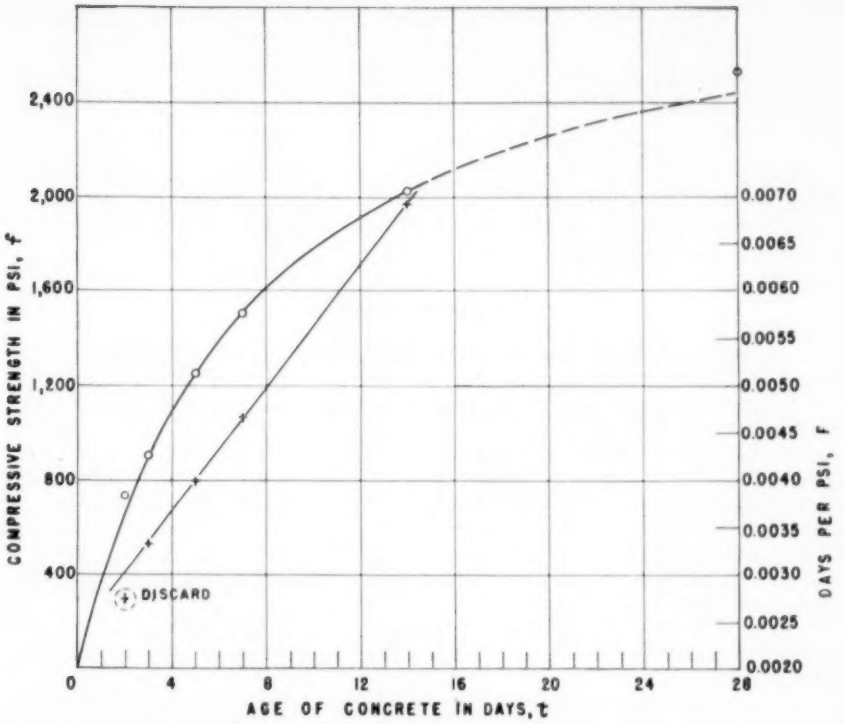

Fig. 2—Computed 28-day strength of concrete by extrapolation of early strength data

The actual 28-day strength was 2534 psi; therefore the error of the estimate is 3.98 percent. This curve is illustrated in Fig. 2.

PRACTICAL TIPS

To eliminate errors which tend to rotate the curves about the later tests, it is important that the time be measured as well as the strength. When the test cylinders are made, the time should be noted on the data sheet. The laboratory must be instructed to record the time at which the tests are made. Then, if the time is expressed in decimal days, to two decimal places, it will be found that the errors in extrapolations will be reduced considerably. The graph paper most suited to this work is 8½ x 11-in. rectangular with 10 divisions to ½-in. each way.

CONCLUSIONS

Each concrete mix, considering such factors as materials, proportioning, placing, curing, and admixtures, has a time-strength relation in which strength is functionally related to time. This functional relation is expressed by Eq. (1). The

treatment of the test data as outlined enables the contractor or engineer to select the correct test data and to recognize those tests which might be in error and should be disregarded. Failure of the function $G(t,F)$ to plot as a straight line serves as a warning that the data are erratic and that the test procedure or some other factor needs correction.

Given the proper test data, the contractor may forecast when subsequent operations may be performed safely. The empirical technique outlined here offers a specification control to the designer and a procedure for programming tests for information and economy.

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For such discussion of this paper as may develop please see Part 2, December 1957 *Journal*. In *Proceedings* V. 53 discussion immediately follows the June 1957 *Journal* pages.

CONCRETE BRIEFS

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Effect of Carbon Dioxide on Fresh Concrete

By EUGENE C. McMILLAN*

The article "Effect of Carbon Dioxide on Fresh Concrete," by J. A. Kauer and R. L. Freeman,† confirms, in almost every detail, something our firm learned the "hard way" 15 years ago. Our experience centered around two jobs. The first involved a 100,000 sq ft area in which a 6-in slab using 2500-psi concrete with a metallic hardened finish was installed. The job was started in the late fall and completed in the early part of December. The weather was such that temporary heat was not necessary. The job finished out with a good hard floor entirely satisfactory to all concerned.

The second job involved about 10,000 sq ft and was the same specification as the first, a 6-in. slab of 2500-psi concrete and a metallic finish. Concrete for this job was furnished by the same company that supplied the first job. The metallic hardener was from the same shipment as that used on the first job and the work was installed by the same crew of mechanics. The only difference was the weather.

This second job was started in January. The first day about 4000 sq ft of concrete were placed. It was extremely cold, and there was no permanent heat in the building, consequently a great number of salamanders were used. So many in fact, the gas concentration was sufficient to send some workmen home sick. By the time the second 4000 sq ft were placed, the weather had moderated and only about half the salamanders were required. The final run of 2000 sq ft was not made until the permanent heating plant was in operation. All of the floors were cured by sprinkling with water.

Results of the variable CO_2 conditions

1. The first placement had an extremely soft, chalky surface that could easily be scratched with a nail to a depth of 1/16 to 1/8 in.

2. The second placement seemed to be as bad in places and the rest of the area was affected and far from a first-class finish.

3. The final placement was hard and a first-class finish in every respect.

To correct the objectionable condition the soft, chalky surface was removed. Beneath it was good, hard, sound concrete.

Reviewing all the facts on the two jobs ruled out defective materials or workmanship and left only the weather and salamander heat as the one difference. The problem was referred to E. W. Scripture of Master Builders Co., Cleveland. He stated that carbon dioxide

*McMillan Floor Co., Hazel Park, Mich.
†ACI JOURNAL, Dec. 1955, *Proc.* V. 52, pp. 447-454.

combines with the hydration products of portland cement forming calcium carbonate which is a soft inert material. The result on floors is a soft, dusting surface. This experience bears out in practically every detail the report by Kauer and Freeman.

Since this experience our firm has maintained the policy of not finishing floors where open salamanders are used and has encountered no further trouble of this nature.

Concrete's Etymological Offspring

By NOCH EIN LESER*

To the growing list† of the "concrete word family" the following 36 should be added. The published list now stands at 346 words that owe their existence, or part of it, to the word concrete. In addition, several words which have previously appeared, are listed with new information.

Word	Where used
All Weather Crete	(Thermoplastic-bonded perlite: rigid insulation) Silbrico Corp., Chicago, Ill.; *Engineering News-Record,* Mar. 22, 1956, pp. 56-58
Alucrete	(Expanded concrete) Australian Patent 159,934 issued to Alucrete Construction, Ltd.; *Ceramic Abstracts,* Apr. 1955, p. 64c
Aristo-Crete	(Blocks) Aristo-Crete, Waterford, Conn.; Norwich telephone directory
Asphalcrete	Thomas Moulding Floor Co., Inc., Chicago, Ill.; *American Builder,* Apr. 1955, p. 222
Bitucrete	(Waterproofing additive—British name for "Hydropel") British Bitumen Emulsion Co., Ltd., Slough, Bucks, England; *Engineer,* Apr. 6, 1956, p. 307
Brickcrete	Beacon Concrete Products, Beacon Falls, Conn.; Waterbury telephone directory
Celcon	(Lightweight roof and floor slabs) Celcon, Ltd., London, England; *Concrete and Constructional Engineering,* June 1955, p. xxiii
Conbex	(Expanding plasticising grouting material) Chemical Building Products, Ltd., Hemel Hempsted, Herts, England; *Concrete and Constructional Engineering,* Jan. 1956, p. cxliii
Concote	(Paint) The Debevoise Co., Brooklyn, N. Y.; *MacRae's Blue Book,* p. 3779
Concretite	1. (Concrete paint) The Garland Co., Cleveland, Ohio; 2. (Integral hardener and waterproofer) Stonhard Co., Philadelphia, Pa.; *MacRae's Blue Book,* p. 3779
Con-Flex	(Blade for cutting green concrete) Champion Mfg. Co., St. Louis, Mo.; *Contractors and Engineers,* Sept. 1955, p. 45
Conplast	(Plasticiser) Chemical Building Products, Ltd., Wandsworth (London), England; *Reinforced Concrete Review,* V. 3, No. 7, 1955, p. xxix

*Just "another reader"—in German.

†Previous lists appeared in ACI JOURNAL, *Proc.* V. 47, Nov. 1950, p. 263, May 1951, p. 752; *Proc.* V. 49, Mar. 1953, pp. 682-684; *Proc.* V. 50, June 1954, pp. 894-897; *Proc.* V. 51, Apr. 1955, pp. 813-817.

Corocrete (Floor protecting covering) Ceilcote Co., Inc., Cleveland, Ohio; ACI JOURNAL, "News Letter," Sept. 1955, p. 32

Crete-Free (Coating for concrete handling equipment) Co-Polymer Chemicals, Inc., Livonia, Mich.; *Engineering News-Record*, June 23, 1955, p. 54

Cretite Calbar Paint & Varnish Co., Philadelphia 25, Pa.; *American Builder*, Apr. 1955, p. 228

Creteplank (Floor and roof planks) Martin Fireproofing Corp., Buffalo, N. Y.; *MacRae's Blue Book*, p. 3785

Drumcrete (Drummed lightweight aggregate) Sullivan Co., Memphis, Tenn.

Febcrete (Air-entraining agent) Feb (Great Britain), Ltd., London and Manchester, England; *Concrete and Constructional Engineering*, Apr. 1955, p. v

Flagcrete (Concrete flagstones) General Concrete Products, Inc., Los Angeles, Calif.; Los Angeles telephone directory

Form-Crete (Device to hold tendons to a predetermined curve in pre-tensioned beams) Food Machinery and Chemical Corp., Lakeland, Fla.; *Engineering News-Record*, Mar. 8, 1956, p. 64

Inlaycrete (Inlaycrete Bond—plastic for bonding concrete to other materials) The Garland Co.; *American Engineer*, Mar. 1955, p. 42

Isocrete Charlton Concrete Co., Ltd., Bath, England; *Concrete and Constructional Engineering*, Jan. 1956, p. cxvi

Konset (Stabilizer) Sullivan Co., Memphis, Tenn.; *American Builder*, Apr. 1955, p. 244

Kontite (Sealer) Sullivan Co., Memphis, Tenn.; *American Builder*, Apr. 1955, p. 244

Mag-O-Crete Camp Co., Inc., Chicago, Ill.; *American Builder*, Apr. 1955, p. 248

Mortar-Crete (Packaged mix) Ready-Crete, Inc., Milwaukee, Wis.; Milwaukee telephone directory

Nucretor (Airplaco Nucretor—concrete gun) Air Placement Equipment Co., Kansas City, Mo.; *Contractors and Engineers*, Mar. 1955, p. 59

Oak Crete Oak Crete Block Co., Milwaukee, Wis.; Milwaukee telephone directory

Patch-Crete (Packaged mix) Ready-Crete, Inc., Milwaukee, Wis.; Milwaukee telephone directory

Perl-Crete Western Mineral Products Co., Minneapolis, Minn.; *American Builder*, Apr. 1955, p. 256

Pre-Krete (Cement lining for water tanks) Patterson-Kelley Co., Inc., East Stroudsburg, Pa.; *Industrial Laboratories*, Aug. 1955, p. 75

Presscon (Precast, prestressed concrete products) Prestressed Concrete of Georgia, Atlanta, Ga., ACI JOURNAL, "News Letter," Dec. 1955, p. 20

Quicksocrete (Accelerator) Chemical Building Products, Ltd., Wandsworth (London), England; *Reinforced Concrete Review*, V. 3, No. 7, 1955, p. xxix

Raphcon	(Contraction of "Reinforced and Prestressed Hollow Concrete Units") Raphcon, Ltd., London, England; *The Engineer*, Apr. 15, 1955, p. 534
Ready-Crete	(Packaged mix) Ready-Crete, Inc., Milwaukee, Wis.; Milwaukee telephone directory
Reconcret	Telegram address for Christiani & Nielsen, Ltd., international contractors, London, England; *Concrete and Constructional Engineering*, Feb. 1955, p. xix
Redicrete	Ready Mixed Concrete Co., Waterbury, Conn.; Middletown telephone directory
Romancrete	General Concrete Products, Inc., Los Angeles, Calif.; Los Angeles telephone directory
Sacrete	Hartford, Conn., *Times*, Feb. 5, 1955, p. 22
Spancrete	(Expanded prestressed precast planks) West Allis Concrete Products Co., Waukesha, Wis.; *Engineering News-Record*, Nov. 24, 1955, pp. 40-41
Suconem	(Admixture) Super Concrete Emulsions, Ltd., Los Angeles, Calif.; Los Angeles telephone directory
Veneercrete	(Concrete additive) Scientific Construction Equipment Co., Milwaukee, Wis.; Milwaukee telephone directory
Vorcrete	Vorac Co., Rutherford, N. J.; *American Builder*, Apr. 1955, p. 274

Below are words which have appeared on previous lists for which new information is presented.

Word	New data
Celocrete	(Concrete block) Devos Block Co., Milwaukee, Wis.
Color-crete	(Block) Angulus Block Co., Burbank, Calif.
Duracrete	Duracrete, Inc., Leesburg, Fla., *Engineering News-Record*, Apr. 14, 1955, p. 13
Jetcrete	Construction Machinery Co., Waterloo, Iowa; *American Builder*, Apr. 1955, p. 242
Konkure	(Concrete sawing and joint filling contractor) Konkure Co., Los Angeles, Calif.
Laticrete	(Flexible concrete composed of liquid rubber and cement) U. S. Rubber Co., New York, N. Y.; ACI JOURNAL, "News Letter," June 1955, p. 37
Novacrete	Nova Sales Co., Trenton, N. J.; *American Builder*, Apr. 1955, p. 255
Red-E-Crete	(Ready-mixed concrete) Red-E-Crete, Inc., Los Angeles, Calif.
Strestcrete	(Floor and roof units) Illinois Brick Co., Chicago, Ill.; *Midwest Engineer*, Oct. 1954, p. 5

Fire Resistance and Permanency of Prestressed Concrete

A question by STANLEY BURNE*

The principles of prestressed concrete are fairly well known and use of prestressing is increasing rapidly. However, there are some doubts as to the integrity of such construction from a long range viewpoint in terms of years of service.

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A building must be considered, to all intents and purposes, a permanent structure. Once completed, it will normally be occupied for years. When a building is constructed of prestressed girders, beams, and other elements, in what condition will it be in 20 years? Questions about tendon slippage, loss of bond, plastic flow and yield of concrete under continuous high stress come to mind. A prestressed girder is stored with energy, in the form of locked-up internal stresses, when it is fabricated; its structural integrity depends on the maintenance and permanence of the conditions under which it was designed and constructed.

How about fire resistance? The writer has seen reinforced concrete structures subjected to fire to the extent that the concrete cover over the reinforcing steel was cracked and spalled, exposing the steel. Yet, such structures did not fail, but were repaired with shotcrete and are still in use. Relatively low tensile and bond stresses, relatively low concrete stresses, and the usual continuity of steel into adjacent construction furnishing ample anchorage offer high resistance to collapse.

Can prestressed concrete be expected to do as well? Or will cracking and spalling of the beam soffit, under heat and the water stream from the fireman's hose, release the tendons and cause collapse? To what extent does the special high-strength steel used for tendons lose its tensile value under heat?

A partial answer abstracted from "Building Research Station Digest" (England), No 84, Jan. 1956

The information presently available on the fire resistance of prestressed concrete structural elements is limited to the results of tests (B.S.476:1953) made on some simply-supported prestressed beams and on floors incorporating prestressed concrete units. From this work some general conclusions can be drawn.

1. The results of fire resistance tests on normal reinforced concrete members cannot be used to assess the fire resistance of prestressed concrete units even of the same size.

2. A system of prestressed concrete construction can be given any grade of fire resistance by suitable design, including protection, of the members. Because of lack of data, it is necessary to err on the side of safety.

Fire tests have shown that the time of collapse for larger sizes of prestressed concrete beams is determined largely by the rise in temperature of the prestressing steel. When hard-drawn wire is used, collapse usually becomes imminent when the wire reaches 400 C (750 F); at about this temperature the strength of the steel is reduced to half its strength at normal temperatures. If the hard-drawn wires in a prestressed concrete beam reach a temperature over about 150 C (300 F), there will be a considerable permanent loss of prestress.

A concrete cover of 2½ in. to the steel will give a fire resistance of 2 hr. Longer periods of resistance may be developed by increasing the thickness of the concrete cover, which should then be lightly reinforced to retain it in position around the cable or cables. For a fire resistance of 4 hr or more, it may be necessary to encase the beam with material of better insulating properties than normal concrete. This may be obtained by encasing a beam having a concrete cover of 2½ in. with a 1 in. thickness of vermiculite concrete, a ⅞ in. thickness of vermiculite gypsum plaster, or a ¾ in. thickness of sprayed asbestos.

When it is known that a prestressed concrete member must have an added protective covering, appropriate steps should be taken during casting of the member, *e.g.*, a preformed covering can be placed against the formwork and cast integrally with the member or, if the member is to be plastered, a good mechanical key must be provided to ensure proper adhesion.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove to be extremely controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.

Q. What is the effect of slump on the compressive strength of concrete with the water-cement ratio maintained constant? Does the common specification requirement for the addition of cement and water, sufficient to main-

TABLE 1—W/C, SLUMP, AND STRENGTH OF CONCRETE MADE WITH NORMAL PORTLAND CEMENT*

W/C, gal. per sack	Cement, lb per cu yd	Concrete		Compressive strength, psi at 28 days
		lb per cu ft	Slump, in.	
4½	607	148.1	1	5870
	638	149.1	2	5760
	660	148.7	2½	5560
	724	149.2	5¼	5830
	772	148.9	7¼	5530
	864	150.6	9¼	5400
	984	149.3	10⅛	5390
5¼	491	143.4	1½	4600
	537	148.4	1¾	4910
	578	149.0	4	4890
	604	149.2	6¼	4910
	666	149.9	9	5060
	733	151.5	9¼	4860
	825	148.7	10	4100
	851	147.8	10	4260
6	431	144.2	¼	3600
	486	147.7	2½	3900
	527	148.0	6¼	4490
	567	149.0	7⅛	3820
	625	149.3	9	3280
	686	147.9	9¾	3100
	747	146.2	10½	3350
6¾	377	142.0	⅛	2950
	399	143.5	½	2860
	454	147.7	5	3270
	504	149.0	7¼	3410
	552	148.5	8¾	2850
	592	148.5	10	2340
	679	148.0	10½	2380
7½	366	144.9	¾	2140
	387	146.5	2¼	2230
	423	146.6	6¾	2470
	463	148.2	7¾	2820
	503	149.8	9	2170
	524	146.2	10½	2240
	571	148.2	11⅛	1810
8¼	346	146.4	¼	2160
	266	146.6	1¾	2200
	399	148.7	5¾	2050
	438	149.8	8⅛	1930
	453	148.9	9	2260
	486	148.5	9½	1820
	506	145.0	10	1680

*Sand and 1½ in. gravel. Ratio of sand to gravel = 1:1.86. Machine-mixed concrete; strengths are average of two 6 x 12-in. cylinders.

tain the same water-cement ratio, maintain the same compressive strength with increases in slump?

A. The tabulations given here present the results of tests in 1931 on this question by the Lone Star Cement Research Laboratory, then under the direction of Duff A. Abrams. This material was made available by Ernst Gruenwald, manager, "Incor" and Technical Service, Lone Star Cement Corp., New York.

The tests included various water-cement ratios, the same sand and gravel aggregates for all mixes, and were performed with cement blended from four standard cements then available on the market. All test cylinders were moist-cured under laboratory conditions. It will be noted that for the particular aggregates used the optimum strengths were obtained in the plastic range (2½- to 6-in. slump). The rich mixes with their high slumps in general produced the lowest strengths for a given water-cement ratio. The very lean, harsh mixes also produced low strength values but this possibly was due to insufficient consolidation provided by the standard rodding procedure.

TABLE 2—W/C, SLUMP, AND STRENGTH OF CONCRETE MADE WITH HIGH-EARLY-STRENGTH CEMENT*

W/C, gal. per sack	Cement, lb per cu yd	Concrete		Compressive strength, psi at 28 days
		lb per cu ft.	Slump, in.	
4½	628	148.3	½	5960
	654	150.1	1	6090
	675	150.8	1¼	5640
	734	150.0	2¾	5830
	780	148.8	4¾	5850
	883	148.8	7¼	6460
	967	148.5	5	6410
5¼	513	144.0	¼	5730
	551	148.1	1¼	6060
	587	149.1	2½	5760
	612	148.7	3½	5870
	665	148.4	5¾	5600
	720	148.7	6¾	5470
	811	148.8	7	5690
	840	148.2	9	4970
6	456	143.0	½	5500
	499	146.2	1¼	4710
	539	148.3	3¼	4900
	570	148.1	5	5000
	621	148.7	8¼	4790
	681	148.7	8¼	5110
	744	147.3	10	4440
6¾	405	144.0	½	4500
	423	146.8	1¼	4650
	473	148.2	4½	4570
	505	148.6	6¾	4600
	550	148.8	9	4340
	586	149.3	8	4210
	661	145.5	10½	3990
7½	383	147.7	1½	3790
	398	146.6	2	4080
	432	148.0	3¼	4110
	465	148.5	6½	3750
	495	148.7	8	3340
	524	148.6	8¼	3370
	559	147.8	9¼	3690
8¼	358	145.0	¾	3700
	375	146.0	1½	3450
	400	147.4	4¾	3610
	433	149.8	6½	3360
	449	152.5	8¼	2900
	479	147.9	8¾	2770
	507	147.9	9¾	2970

*See footnote to Table 1.

Q. How may one avoid flooring trouble due to moisture in floors of basements and slabs on grade?

A. Solid concrete floors can never be relied on to act as complete barriers to passage of water from the ground, whether in liquid or vapor form. Ground moisture can pass through them until it either disperses by free evaporation at the surface or reaches a less permeable material under which it tends to accumulate. The suitability of a floor covering, if rising dampness can reach it, is judged by its ability to transfer moisture harmlessly to the air above as fast as it accumulates below; or, if it cannot do this, by its tolerance to moisture without decay, distortion, or loss of adhesion.

The difficulties due to moisture are caused by the use of a floor surfacing material not resistant to moisture in combination with a floor construction lacking a barrier to moisture. Elimination of the trouble in planning a floor on a wet or moist base may be accomplished through use of an absolutely impenetrable barrier to passage of water, whether liquid or vapor. (Integrally waterproofed concrete will resist penetration of liquid water through capillaries but not water vapor.)

In areas of year-round dryness with little recorded rainfall, the moisture barrier may not be needed. ACI Committee 332, Recommended Practice for Residential Concrete Work, considers that such omission of vapor barrier should depend upon local experience and ample technical data to justify it.

Materials are readily available which meet applicable requirements for an effective vapor barrier. The material should have vapor and moisture passage resistance and also be resistant to traffic puncture and deterioration. The barrier material should be equivalent to, or be 55-lb roll roofing. Acceptable types of polyethylene film meet this equivalent requirement and have good performance experience. Where vapor barrier only is needed, edges of material may be lapped without adhesive. If the material must also serve as waterproofing, edges should be lapped with adhesive.

Committee 332 expects to publish soon its report "Slabs on Ground for Residential Use," giving recommended details for installation of such barriers within or below the slab. Incidental benefits of such construction are free choice of floor finish and most efficient operation of thermal insulation under the floor.

The second obvious remedy, applicable to existing floors not dampproofed, as well as new construction, is the use of a finish floor surface not affected by the moisture. Such a finish should be capable of resisting or transmitting rising dampness without dimensional, material, or adhesion failure. Examples of such floor finish are: (1) for industrial floors—asphalt mastic laid on hot mopped surface, exposed concrete finish, portland cement terrazzo, and clay tile; (2) for residential, school, and institutional floors—the latter three. Wood blocks and asphalt or vinyl tile are capable of partially transmitting rising dampness without material failure and generally without dimensional or adhesion failure. Rubber tile is impervious but prone to adhesion failure. Magnesite terrazzo, linoleum, carpet, cork, and wood surfaces transmit dampness but are sensitive to moisture, and dimensional, material, or adhesion failure results.

The above precautions are unnecessary where naturally dry conditions exist and may be inadequate where an active head of water pressure exists. The degree of protection economically justified depends in general upon immediate site conditions and usually involves some "calculated risk."

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Prestressed concrete for world's longest vehicular bridge

J. J. WADDELL, *Rock Products*, V. 59, No. 4, Apr. 1956, pp. 233-236

The 24-mile long Lake Pontchartrain Bridge consists of a precast monolithic deck and prestressed beam units resting on concrete caps which are in turn supported by 54-in. diameter hollow prestressed concrete piles; two piles to a bent. Bents are spaced 56 ft on centers. Precasting and prestressing operations are described. The deck units are precast in one piece complete except for the aluminum hand rail and carried to the site of the bridge by barge.

Standardized sections are the key to a fast, economical bridge job

Contractors and Engineers, V. 53, No. 5, May 1956, pp. 40-42

For the 947-ft Eureka Slough bridge in northern California, all units except the pile caps are precast. The substructure consists of precast, pre-tensioned concrete piles with cast-in-place pile caps. The approach spans, 30 ft long, are composed of precast, pre-tensioned 4 ft wide units; 252 deck unit slabs are required. The navigation span consists of 105-ft post-tensioned, prestressed girders, T-shaped so that the top flanges will create the deck area.

Construction

Prefabricated exhibition hall in Stockholm

Prefabrication (London), V. 3, No. 30 Apr. 1956, p. 271

Exhibition hall for Swedish motor show has 110,000 sq ft floor area and was constructed in 9 weeks with a labor force of 30. It consists of frames of prestressed concrete enclosed by wall and roof units of standard size prefabricated Siporex building units. The units did not require plastering or stucco on the exterior. They were painted with a water-repelling paint. The record time of construction was achieved in spite of the government requirement that no worker be under 55 years of age and in spite of winter conditions.

Sixty-six precast bays go up in 11 days

Engineering News-Record, V. 156, No. 21, May 24, 1956, pp. 44-46

In the erection of a precast three-story building precast one-piece columns 38 ft long were used to support precast, pre-tensioned stems for T-beam girders which carried precast channel floor and roof elements 2 ft wide and 20 ft long. The columns incorporated steel base plates and were erected and fixed similarly to steel columns.

Producing cooled concrete

Contractors and Engineers, V. 53, No. 5, May 1956, pp. 152-153

Describes the use of ice-produced cool concrete on a small project in Philadelphia. It was found that the use of 50 lb of ice per cu yd reduced the temperature of the concrete 20F at the plant. The estimated additional cost on this small project involving 850 cu yd was $1 per cu yd.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 28, No. 2, August 1956, *Proceedings* V. 53. Address 18263 W. McNichols Rd., Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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84 tons of girder sets record

Engineering News-Record, V. 156, No. 21, May 24, 1956, pp. 51-54

Ten 136-ft precast post-tensioned girders were successfully positioned on the roof of a hangar at Hill Air Force Base, Ogden, Utah. The 84-ton units are believed to be the largest concrete girders precast for a building. They are joined integrally with supporting columns to provide a rigid frame structure. The roof itself consists of precast ribbed roof slabs. The precast prestressed rigid frame design was selected as the most desirable of several alternative framing systems, including laminated wood arch, laminated wood arch on concrete buttresses, structural steel trusses, precast concrete three-hinged frame, precast prestressed concrete simple span girder, precast prestressed concrete frame, and prestressed concrete shell.

Traveling steel forms speed lock construction

R. MONSON, *Contractors and Engineers*, V. 53, No. 5, May 1956, pp. 28-33

Concrete on Lock No. 19, Mississippi River, Keokuk, Iowa, was placed at a rate of 1200 cu yd per day. The production rate is attributed principally to a novel set of traveling steel forms developed for this project. The wall forms are 30 ft long, as high as the lift requires, and consist of the two sidewall forms and a bulkhead. After casting, jacks are used to lift and tilt the form to clear dowels and move ahead to a new position on rails located on the top of the wall. Three sets of such forms in echelon allow work to progress on three lifts of the wall simultaneously.

Casting, handling methods hurry work on big store

Contractors and Engineers, V. 53, No. 5, May 1956, pp. 110-111

A three-story department store building employed round, precast, prestressed concrete columns; precast, prestressed concrete girders; and precast reinforced concrete deck slabs of the channel type. The exterior shell of the ground floor employed cast-in-place conventional reinforced concrete walls.

Hydraulic jacks lift slipforms to build monolithic bridge piers

R. DAY, *Contractors and Engineers*, V. 53, No. 6, June 1956, pp. 6-11

In construction of the 210-ft high piers for a new bridge across the Pecos River near Langtry, Texas, the slipforms are raised by hydraulic jacks. Two changes of section were involved, the final change from a massive base to a two-column bent with a heavy cap. Gives complete details on the forming, placing, concrete materials, and details employed.

Design

Application of an electronic digital computer to some problems of structural analysis

R. K. LIVESLEY, *The Structural Engineer* (London), V. 34, No. 1, Jan. 1956, pp. 1-12

AUTHOR'S SUMMARY

Describes a method of carrying out elastic structural analysis, based on the electronic computer installed at Manchester University. A computer program is described which will analyze automatically a wide variety of plane structures, in each case taking into account both the direct axial strains and the moments produced by axial forces. Examples are given of structures which have been analyzed. The paper also discusses some of the practical points which arise in the solution of engineering problems on digital computers.

Economy in design and construction of industrial roof structures of long spans using the principle of precast prestressed concrete (in Rumanian)

M. HALMAGIU, *Industria Constructilor si a Materialelor de Constructii* (Bucharest), V. 6, No. 1, Jan. 1956, pp. 50-57

Reviewed by J. J. POLIVKA

New method of assembling triangular trusses of standardized rectangular precast concrete frames with tensile steel diagonals is described and its advan-

tages, especially economy, discussed on the basis of an executed structure spanning 82 ft. Individual panels 6 ft 9 in. long and 10 ft high are tied together by stressed cables, inserted in the center holes of chords, consisting of ten 0.2-in. wires in the upper chord, and 48 wires (0.2 in.) in the lower chord. Russian (Korovchin) prestressing method was used. Tests with two assembled trusses were carried out, with stresses and deflections measured under increasing loading. Costs are compared with those of similar standard trusses on the basis of amounts of materials required per square foot of roof coverage (trusses only). Results of this comparison are:

Material (per sq ft of coverage)	Standard precast truss	Prestressed truss
Concrete (average thickness), in.	5.8	3.4
Ordinary reinforcement (including rolled shapes), lb	9.8	2.2
Prestressed wires (cables), lb	-	0.3

Of interest are amounts of materials of similar trusses designed by author, spanning 165 ft and arranged 32 ft apart: concrete—2.4 in., steel—4.2 lb, prestressed cables—0.9 lb.

Slide rule for concrete beam computations (Aristo-stahlbeton-system Göttsch)

A. S. Schoevers, *Polytechnisch Tijdschrift* (The Hague), V. 11, No. 7-8, Feb. 16, 1956, pp. 123b-126b

Reviewed by John B. Snethlage

Describes and explains, with sample computations and sketches, a special slide rule, which enables the making of all computations for reinforced concrete beams, rectangular and T-shaped, without the use of tabulations. No mention is made of stirrups. The decimal system is used and the slide rule is based on n=15. Others, for n=8 and n=10, will soon be available.

Ultimate load method of design

A. L. L. Baker, *Concrete and Constructional Engineering*, (London), V. 51, No. 3, Mar. 1956, pp. 293-302

Discusses the selection of safety factors, their derivation in a logical relation to structural conditions, and the desirability for consistent safety factors throughout a structure. Proposes a system for the determination of safety factors in which weighted values are given to such factors as the results of failure; type of workmanship; conditions of load; importance of member; warning of failure; possible reduction of strength through use; and accuracy of analysis.

Design and construction of barricades

R. L. Porter, P. A. Lobo, and C. M. Sliepcevich, *Industrial and Engineering Chemistry*, V. 48, No. 5, May 1956, pp. 841-845

Describes the requirements for the design of a barricade to protect adjacent personnel and property from nuclear radiation. Material for these barricades may vary from boiler plate to reinforced concrete.

Analysis of frames with two gables

V. A. Morgan, *Concrete and Constructional Engineering*, (London), V. 51, No. 2, Feb. 1956, pp. 265-272

A general solution for moments and thrusts on double gable frames. The paper is essentially a continuation of an earlier paper (July 1955) in which the general solutions were given for single gable frames.

Stress analysis and design of foundations for machines and turbines (Gep- és turbinaalapok számitása és tervezése)

S. Major, State Publisher for Technical Books, Budapest, Hungary, 1956, 668 pp., 136 Hungarian Florints

Reviewed by J. Barta

Book is useful both as a textbook and a handbook for structural engineers. Subjects include: theory of vibrations; building materials for machine foundations; soil testing; analysis and measuring of dynamic stress of soils; structural description and stress analysis of foundations for steam hammers, crank-

shaft driving machines, turbines, and other machines; methods and equipment for elimination of vibrations. Detailed numerical examples are presented for those cases most frequently encountered in practice.

Materials

Performance test for the potential sulfate resistance of portland cement

ASTM COMMITTEE C-1. *ASTM Bulletin*, No. 212, Feb. 1956, pp. 37-44

Reports on a cooperative series of tests to develop a performance test for potential sulfate resistance of portland cements. The test consists of measuring expansion of mortar prisms containing excess calcium sulfate. The expansion of mortar prisms is an index of the potential sulfate resistance of the cement. Results indicate that: (1) results may be evaluated in 28 days; (2) results are reproduceable; (3) expansions discriminate between different cements; (4) there is a relation between expansions and C_3A content of cement, and (5) results can be correlated with performance of concrete exposed to sulfate soils.

Some observations on the mechanics of alkali-aggregate reaction

L. S. BROWN, *ASTM Bulletin*, No. 205, Apr. 1955, pp. 40-46 (including discussion)

Presents observations, on various concretes showing distress, by microscopical examination. Includes some 18 plates, picturing the symptoms of distress on microscopic scale, and concludes with a general discussion of the difficulties encountered in such an examination as well as some conclusions as a result of this work.

An improved sulfate soundness test for aggregates

D. O. WOOLF. *ASTM Bulletin*, No. 213, Apr. 1956, pp. 77-84

AUTHOR'S SUMMARY

The present sulfate soundness test, AASHO T 104-46 or ASTM C 88-46T, permits in tests of coarse aggregates the determination of six different values, anyone of which may be considered as "standard." Different laboratories may not agree on the method to be used in testing a given sample of aggregate. To correct this condition, it is recommended that the method be used for the acceptance but not the rejection of aggregates; that the sample tested be restricted to one size of coarse or fine aggregate; that the drying oven meet requirements for performance; that the loss of coarse aggregate be determined with sieves having openings one-half the size of those in the original retaining sieves; and that certain minor changes be made.

Increasing the plasticity limit of constructional steel in reinforced concrete constructions (in Polish)

J. KOBIAK. *Inzynieria i Budownictwo* (Warsaw) No. 11, 1954, pp. 330-343

POLISH TECHNICAL ABSTRACTS No. 4(20), 1955

Increasing the plasticity limit of reinforcement makes it possible to effect considerable savings. Purpose of the test described was to investigate possibility of increasing the plasticity limit during construction by straightening the bars delivered in bundles. For this purpose a special device was developed; results obtained were satisfactory.

Influence of the addition of slag on the strength of mortars manufactured from ground quicklime (in Polish)

W. WLOSIK. *Cement, Wapno, Gips* (Stalinogrod), No. 10, 1954, pp. 1-4

POLISH TECHNICAL ABSTRACTS No. 3(19), 1955

Investigations show that the best effect on bending, compressive, and tensile strength of quicklime mortar was obtained by adding 10 to 15 percent ground slag.

Pavements

Asphalt or concrete?—Oklahoma builds test project to compare

Roads and Streets, V. 99, No. 4, Apr. 1956, pp. 114-119

Test project near Oklahoma City consists of a four-lane dual highway incorporating parallel divided roadways of

portland cement concrete and asphaltic concrete. Each type is represented by a pavement, two traffic lanes wide and about 4 miles long. The test was initiated by a joint resolution of both houses of the Oklahoma legislature because they questioned the economy in the widespread use of asphaltic concrete due to its "tremendously high maintenance expense."

Lives of highway surfaces—half-century trends

G. D. GRONBERG and N. B. BLOSSER, *Public Roads*, V. 29, No. 2, June 1956, pp. 17-28

The major objectives of the road-life study phase of the highway planning surveys is the development of factual data relating to (1) how long road surfaces remain in service before they are replaced; and (2) what is done to the road surfaces at the time of replacement. This is the third report in the past 16 years (the first, in 1941; the second, in 1949).

Precast concrete

Experimental system for increasing productivity

J. G. BUITINK, *Prefabrication* (London), V. 3, No. 30, Apr. 1956, pp. 268-270

Describes erection and design of a precast building system developed in Holland. Precast beams and columns are employed which are joined by vacuum treated grout and covered by cast-in-place floors. The principal saving is that of two-thirds of the carpentry work.

Joints in precast frames

C. H. DOBBIE and R. L. WAJDA, *Concrete and Constructional Engineering* (London), V. 51, No. 4, Apr. 1956, pp. 345-350

Presents results of tests on the strength of various joints developed for use in precast concrete frames. These joints are bolted, eliminating the need for grout and welding of reinforcement. It was concluded that the compressive strength of the concrete controls the strength of such joints; that a scarf joint similar to that used in timber, which provides a measure of interlocking, considerably increases the over-all strength of the joint provided that connecting bolts are tightened sufficiently. The tests show that these joints are rigid and will safely resist a considerable bending moment.

Prestressed concrete

Report on present practice regarding grouting and anchorages in prestressed concrete in Great Britain

A. W. HILL. *Paper* No. 8, Session Ia, Second Congress of Federation Internationale de la Precontrainte, Amsterdam, 1955

AUTHOR'S SUMMARY

The functions of grouting in post-tensioned prestressed concrete members are briefly reviewed and grouting methods and plant used in Great Britain are summarized. Site problems are discussed together with the properties of the hardened grout in relation to volume change, durability, and bond properties as shown by tests carried out to destruction. No problems have arisen in relation to corrosion or to exposure to high temperatures, but in frosty weather care is necessary and air entrainment may be an advantage.

General requirements of a satisfactory anchorage are stated, and the types in use in Great Britain are discussed in relation to the stress distribution on the adjacent concrete, loss in stress due to slip and friction. Recent research in these subjects is briefly described.

Problem of using chord steel in piles and compressed elements of prestressed concrete (in Polish)

T. KUCZ, *Inznieria i Budownictwo* (Warsaw), No. 12, 1954, pp. 366-376

POLISH TECHNICAL ABSTRACTS No. 4(20), 1955

Presents the results of tests, discussion of savings, and advantages accruing from this construction. Tests with concrete piles reinforced with wood, prestressed inserts have revealed that full cooperation is possible between strands 1.5 mm in diameter and concrete of at least 4500 psi compressive strength in axially compressed piles of concrete reinforced with prestressed inserts. The same applies in full sections of prestressed piles.

Relaxation of high tensile strength steel wire for use in prestressed concrete

G. McLean and C. P. Siess, *ASTM Bulletin*, No. 211, Jan. 1956, pp. 46-52

Tests were made to investigate the effects of (a) type of wire; (b) stress level, and (c) preliminary overstressing. A test method for measuring relaxation of highly stressed steel wires is described and typical results are presented from exploratory tests on 28 specimens of four types of wires. Stress losses are shown up to 1000 hr in all tests and for some up to 5000 hr. A preliminary indication is that at stresses higher than 70 percent of the ultimate, the losses are significantly greater for the stress-relieved wire. At these higher stresses, the loss may be reduced considerably by initial overstressing.

Prestressed silos for sugar

Concrete and Constructional Engineering, (London), V. 51, No. 3, Mar. 1956, pp. 309-312

Two silos, interconnected for storage of 16,000 tons of refined sugar, are 66 ft in diameter and 100 ft high, built on reinforced concrete columns to provide a basement for conveyors. The silos were constructed by the conventional slipform method. Prestressing was accomplished by the Gifford-Udall system in which groups of separate wires are anchored. Each wire unit extended half around the circumference.

Prestressing in Germany (in Italian)

Giorgio Neumann, *Rassegne dei Lavori Pubblici*, V. 2, No. 12, Dec. 1955, pp. 503-507

Reviewed by J. J. Polivka

Emphasizes that every new construction method makes it necessary to follow progress achieved in other countries. Author mentions basic work of Freyssinet (France); Koenen, Dischinger, Moersch, Finsterwalder, Ruesch, Leonhardt (Germany); and Emperger, Wettstein (Austria). Most spectacular structures in Germany are described and illustrated, with comments on their original and progressive features, *e.g.*: railroad bridge over Neckar River near Heilbronn; Rosenstein bridge over Neckar River, Stuttgart; bridge over Ruhr River, Froendenberg. Basic principles of design and construction methods and properties of materials are described, discussed, and compared with those employed in Italy.

Initial attempt at using prestressed concrete in hydro-engineering constructions (in Polish)

P. Slomianko, *Technika i Gospodarka Morska* (Gdansk-Wrzeszcz), No. 3, 1954, pp. 82-84

Polish Technical Abstracts No. 3(19), 1955

Reports results of experiments on watertight wall elements made with chord-prestressed concrete. The elements were tested by driving them in with free fall and high-speed drop hammers and subjected, after being extracted, to bending. Behavior of the elements during the experiments tends to prove that further savings in steel in chord-prestressed concrete elements can be made by reducing the amount of transverse reinforcement.

Prestressed concrete pavements (in Spanish with English, French, and German summaries)

A. Saenz de Heredia, *Publication* No. 164, Instituto Technico de la Construccion y del Cemento (Madrid), 1955, 82 pp.

Reviewed by E. Rosenblueth

Enthusiastic and comprehensive exposition of prestressed concrete as used in several countries for highway, airport, and warehouse and pavement work. Includes a realistic appraisal of the advantages and drawbacks of the system; a sketchy but sufficiently self-contained description of methods for computing stresses due to temperature, shrinkage, and load, and loss of prestress; a survey of prestressing techniques, both with cables and fixed abutments, contrasting clearly the practice of Leonhardt, Freyssinet, Roeblings, etc.; and a survey of all major works of this sort executed throughout the world. The paper is a welcome addition to the literature on a system that has thus far been approached in an excessively timid and tentative spirit.

Long prestressed girders frame roof of gymnasium

R. Monson, *Contractors and Engineers*, V. 53, June 1956, pp. 52-56

Describes and pictures the forming, prestressing, and erection of the 138-ft

span prestressed roof girders for the gymnasium of the Parkview High School, Springfield, Mo. The girders are said to be the longest precast, prestressed building girders constructed and erected in this country. The 104-ton girders were lifted by hydraulic jacks mounted on the top of twin columns between which the girders were precast. Short cables were later inserted through column and girder ends to provide rigid frame continuity for live load.

Rules for the design of prestressed concrete

D. V. Isaacs and G. W. Anderson, *Special Report* No. 15, Commonwealth Experimental Building Station, Sydney, Australia, Aug. 1955, 16 pp., 3 shillings

A brief performance-type code for the design of prestressed concrete structures and structural elements prepared for the City Council of Sydney. It covers construction specifications and design procedures, including allowable stresses through the various stages of construction, load factors, composite (precast concrete to cast-in-place concrete) design, and design provisions both for ultimate strength design as well as design by the elastic theory. It should be a useful guide to designers and manufacturers of prestressed concrete in the United States in the interim until completion of the report by the joint ACI-ASCE committee on prestressed concrete.

Prestressed concrete superstructure

F. E. Koebel and B. Sonesson, *Civil Engineering*, V. 26, No. 3, Mar. 1956, pp. 52-57

A prestressed concrete bridge will replace the flood-wrecked International Bridge at Laredo, Texas. The bridge consists of two approach spans and five 150-ft principal spans. The old bridge consisted of five 150-ft span, three-hinged arches supporting a two-lane roadway. The new bridge, which will consist of prestressed concrete girders with shop-installed prestressing made continuous in the field by field-installed cap cables, will support a four-lane road utilizing existing foundations with no increase in total dead weight over the original two-lane bridge. Design details and drawings are given.

Properties of concrete

Foaming agents for foam concrete (in German)

Johannes Pfanner, *Silikattechnik* (Berlin), V. 6, No. 9, 1955, pp. 396-398

Ceramic Abstracts Apr. 1956 (Hartenheim)

To produce highly porous concrete without the necessity of hardening, foaming agents are added to the usual concrete mixture; they consist of soaps, saponins, sulfonates, and albumen products. Fine sand or preferably quartz meal, Amorphite, and Sikalite (of East German origin) should be used as aggregates. The addition of stabilizers, such as polyvinyl alcohol, waterglass, starch solution, or cellulose glycolate solution, is advantageous. A typical batch (in parts by weight) is 180 portland cement, 180 fine sand, 150 water, and 5 foaming agent (ammonium fatty acid soap + alkylnaphthalene-sulfonate); flexural strength is 20.4 kg per sq cm, compressive strength 52 kg per sq cm, and density 0.94 kg per cu dm. This cold-setting method is economical but requires high-capacity mixers; the finished pieces must be stored about 6 months before use to let them reach the required low shrinkage.

New method of measuring the tensile strength of concrete and other brittle materials (in Polish)

W. Olsak, S. Kajfas, and J. Pietrzykowski, *Inzynieria i Budownictwo* (Warsaw), No. 9, 1954, pp. 271-277

Polish Technical Abstracts No. 4(20), 1955

Describes a method of measuring tensile strength of concrete whereby testing cylinders are crushed between two parallel surfaces. Includes results of measurements demonstrating linear relationship between compressive strength and tensile strength. (This is apparently the method developed in Brazil. See "Current Reviews," ACI Journal, July 1956, *Proc.* V. 53, p. 139.)

Poisson's ratio of concrete: A comparison of dynamic and static measurements

J. C. SIMMONS, *Magazine of Concrete Research* (London), V. 7, No. 20, July 1955, pp. 61-68

AUTHOR'S SUMMARY

Static and dynamic tests were carried out on a series of concrete specimens to determine the values of Poisson's ratio. Three values of the ratio were determined, the static value and two values derived from the dynamic tests. Where possible the values are related to each other and to other properties of the concrete. Values of the elastic moduli determined from both types of tests are also compared. From the results of the dynamic tests an equation is derived by which the dynamic elastic modulus may be determined from measurements of the ultrasonic pulse velocity.

Structural research

Behavior of composite lintel beams in bending

A. M. OZELL and J. W. COCHRAN, *Journal of Prestressed Concrete Institute*, V. 1, No. 1, May 1956, pp. 38-48

Reports on tests of composite beams consisting of U-shaped bottom elements, prestressed with cast-in-place top cores. Includes load test data which substantiate the conclusions that assumption of elastic behavior for composite beams is valid and that natural bond between the bare beam and the cast-in-place portion provides reliable resistance to horizontal shear.

Effect of varying the pre-tension and area of reinforcement on the ultimate bending strength of concrete beams

J. M. PRENTIS, *Magazine of Concrete Research* (London), V. 7, No. 21, Nov. 1955, pp. 143-150

AUTHOR'S SUMMARY

Tests on 18 pre-tensioned concrete beams are reported. Beams were cast in groups of three, the members of any one group differing only in the amount of pre-tension. The percentage of reinforcement varied between groups but was always sufficient to preclude failure by steel rupture. The beams were all 4 in. deep with an effective depth of 2½ in. Two widths were used: 1⅛ in. and 1½ in. The length between supports was 45 in. and under four-point loading the central 15 in. was subjected to a uniform bending moment.

Both the area of steel and the degree of pre-tension were found to have an appreciable effect on the bending strength of the beams.

General

Practical method of concrete mix design

L. BOYD MERCER. Reprint from *Concrete*, Melbourne, Australia, 1955

A method of calculating mixes, "practical" in that it is intended for the production of controlled quality concrete from available materials, consists of practical means for the utilization of knowledge based on research in materials and theory of statistical control, particularly applicable to control operations for permanently estblished ready-mixed concrete plants. Bulletin carries mix calculations through each step from specification to finished product. The explanation is clarified by a large number of curves and charts. In addition, a number of standardized forms are given which are suggested for use by ready-mixed concrete companies or similar operations for control of their concrete proportioning.

Suggested specifications for expanded shale, clay, and slate concrete masonry construction

Expanded Shale Concrete Facts, V. 3, No. 2, pp. 10-12

Presents producers' recommended specifications for concrete masonry construction using their product in normal construction and also in construction to be earthquake or hurricane resistant. The specifications include materials, resign, and workmanship. Provisions to prevent contraction cracking include bond beams and horizontal joint reinforcing. The use of control joints is not mentioned.